SANITATION IN SOUTH AFRICA

by

NICHOLAS BENECKE

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Supervisors: Mr. C. Loubser, Mrs. A. Bosman

Faculty of Engineering

Department of Civil Engineering

Division of Water and Environmental Engineering

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Declaration

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Signature: Nicholas Benecke

March 2021

Date

ACKNOWLEDGMENT

All glory to the sea urchin. That most curious and courageous of Lord Poseidon's subjects.

SANITATION IN SOUTH AFRICA

Abstract

by Nicholas Benecke, MSc
Stellenbosch University
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The state of South African infrastructures is in decline, yet the degree of decline is neither well quantified nor well understood. Specific to the infrastructural field of sanitation in South Africa, a framework of data analysis and uncertainty analysis was developed in order that logical arguments could be deduced. Due to South African specific conditions which have radically altered the social and infrastructural systems of the country, a holistic coverage of this topic does not exist in literature. Through presentation and analysis of macro-patterns of decay in the nation's most efficacious sanitation system (Cape Town), coupled with a technical analysis of a key macro-pattern (mechanics of sedimentation) a concise argument is presented in an attempt to define an otherwise indistinct problem. This argument was formulated through case study analyses in combination with an exhaustive uncertainty analysis of the theory of the mechanics of sedimentation. Through the application of Systems Thinking and Theory as well as complex analytics, this research posits that sanitation systems in South Africa are experiencing heavy duress and are likely failing to deliver requisite efficacy.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

List of Abbreviations and Symbols

Abbreviations

AI - Artificial Intelligence

BOD - Biochemical Oxygen Demand

CRISP DM - Cross Industry Standard Process for Data Mining

DHS - Department of Human Settlements

DO - Dissolved Oxygen

DS - Dissolved Sulphides

DWAF - Department of Water Affairs and Forestry

DWS - Department of Water and Sanitation

E. Coli - Escherichia Coli

GLS Consulting - Geustyn, Loubser and Streicher Consulting

GDWQ - Guidelines for Drinking Water Quality

GDP - Gross Domestic Product

GIS - Geographical Information System

HF - Heavy Filtered Data-set

ILOST - Immutable Law of Societal Termination

LF - Light Filtered Data-set

ML - Machine Learning

SSD - Simple Sewer Design

SANGWQ - South African National Guidelines to Drinking Water Quality

WHO - World Health Organisation

WTP - Water Treatment Plant

WWTP - Wastewater Treatment Plant

Symbols

D - Diameter [m]

d_{50} - Median Particle Size [mm]

D_* - Effective Particle Diameter

f - Darcy-Weisbach Friction factor

g - Gravitational Constant [m/s^2]

K - Cohesiveness Constant

k - Absolute Roughness [m]

L - System Load

M - Erosion Rate Constant [kg/m^2]

r - Correlation Coefficient

p_s - System Reliability

p_f - System Failure Probability

P_r - Probability of Resistance

R - System Resistance

R - Hydraulic Radius = Flow Area/Wetted Perimeter $[m]$

S - Slope $[m/m]$

s - Specific Gravity $[dimensionless]$

SF - Safety Factor

SM - Safety Margin

U^* - Shear Velocity *[]m/s]*

V - mean flow velocity $[m/s]$

w - Particle Fall Velocity $[m/s]$

ρ - Fluid Density $[kg/m^3]$

ρ_s - Sediment Bulk Density $[kg/m^3]$

ν - Kinematic Viscosity of Fluid $[m^2/s]$

τ_c^* - Shields Critical Shear Stress (obtained from the Shield's diagram) $[N/m^2]$

τ_s - Sediment Bed Shear Strength $[N/m^2]$

τ_0 - Boundary Shear Stress $[N/m^2]$

Dedication

This thesis is dedicated to my father, Hans Christian, my mother Michele Francoise Christine, My brother Paul Christian and my sister Josephine.

To you all, I owe more than I care to contemplate.

Das Leben ist schön, aber teuer - man kann es auch etwas billiger haben, aber dann ist es nicht mehr ganz so schön.

Chapter One

Introduction

1.1 Background of Sanitation

It has been argued that historically more lives have been saved by Sanitation Engineers than all Medical Practitioners. This is a hard argument to prove quantitatively, since there are generally records of lives saved by medical practitioners, but no records will have been created of humans who did not become sick and die as a result of having effective sanitation. However, the qualitative logic sustaining this argument is that prevention beats the cure.

It is generally acknowledged that effective sanitation will improve the standard quality of health of a community of people, since effective sanitation strongly impacts on the control and eradication of infectious and other communicable diseases (Van Zyl and Van Dijk, 2011).

A study conducted in 2012 found that there is a 2:11 cost benefit ratio of improvement of sanitation versus societal damages (World Health Organisation, 2019). In South Africa this translates as, for every R 1.00 spent on sanitation improvement there will be an R 5.50 saving in health related damages. This financially justifies the aforementioned argument of prevention beating the cure.

However, it has been said that people these days know the price (quantitative) of everything, yet the value (qualitative) of nothing (Haldane, 2012). What is the value of sanitation? Water is life and sanitation is dignity (Department of Water Affairs and Forestry, 2003) furthermore it is the absence of a serious risk of death or disability caused by waste-borne disease.

Irrelevant of the specific historical time-related cost of sanitation it has always been qualitatively understood that effective sanitation is an essential part of a functional society. Archaeological findings have shown that formalised sewerage structures were being designed and implemented in the ancient world from 3750 B.C. in Nippur to 2600 B.C. in Tell Asmar to 800 B.C. in Rome (Van Zyl and Van Dijk, 2011).

The aforementioned sewer in Rome was called the Cloaca Maxima, meaning 'Greatest Sewer'. It predated the first roman aqueducts by approximately 500 years (Van Zyl and Van Dijk, 2011). This clearly illustrates the importance, which the ancient Romans placed on effective sanitation.

It is interesting to note that cloaca means 'sewer' in Latin; however, cloaca is also the name given to the combined (intestinal, urinary and genital) excretory orifice of many relatively simple vertebrates. There is an interesting parallelism between the general excretion of bodily fluids from these simple vertebrates and the general excretion of societal human waste both through the 'cloaca'.

In ancient Rome most waste was dumped into a simple channel to be removed by the flow of water to a place (usually the nearest river system) where it would not sicken the collective body of the roman society. In many vertebrates, most waste is dumped through a simple collective orifice in a largely fluid form, preventing waste build up and associated toxicity in the body of the vertebrate.

The common denominator of these two very different types of 'body' is a simple mechanism for waste disposal, a pathway through which largely fluid waste is disposed of to maintain the health of the body. The common result is the continued health of the body.

Considering that parts of the original Cloaca Maxima are still functioning as elements of modern Rome's sewer network (Van Zyl and Van Dijk, 2011) an interesting question is raised of how can an approximately 2800 year old sanitation system still be functional in modern society. The essential answer to this is simplicity of design.

Simple design is not to be confused with rudimentary design. Rudimentary design manifests from a lack of suitable resources to complete a fully functional design. So a subpar design is chosen in lieu of no design. Simple design is the irreducible minimum of design components, which achieve a specified goal. Where rudimentary designs often fail due to lack of robustness, simple designs have a reputation for a high degree of robustness and as such they rarely fail to deliver the planned functionality.

This robustness and value of simplicity has been physically illustrated in such cases as the still functional structures (roads, aqueducts, viaducts and sewers) of ancient roman civil engineers and the nigh indestructibility of the micro-animal the tardigrade. Furthermore the renowned Leonardo Da Vinci has elegantly phrased it, as "Simplicity is the ultimate sophistication".

Considering that civil engineering is arguably one of the most relevant fields of applied sciences to the real-world, ensuring that the theoretical work of civil engineers is ultimately applicable in reality is an absolute necessity.

Humanity excels at perceiving in causality and under-performs in perceiving in statistics (Kahneman, 2012). It is well known that metaphors are invaluable aids in simplification of explanation and understanding-transfer of complex thought systems from an author to a reader.

To this end the metaphor of a given society as a unitary organism will be used. Specifically the concept of the alimentary canal of an organism as it relates to the consumption (relatively indiscriminate ingestion), digestion (selective extraction) and then excretion (waste dumping) of energy and matter by a societal 'body'.

1.2 Problem Statement

South Africa has amongst the highest air pollution rate, ranking at 15th in the world (World Population Review, 2019), highest crime rates ranking 3rd in the world (Numbeo, 2018) and the highest inequality rates in the world (Indexmundi, 2015) amongst a plethora of other negative global rankings. All of this leads to an increasingly fractionated society with an increasingly limited capability of unifying to achieve quantitative and qualitative improvements for the majority of the citizens of the country.

The results of this are hard to quantify in the social sphere; however, the damage to infrastructure can be quantified to a significantly higher degree of certainty.

An interested observer of water affairs in South Africa will quickly realise that something has gone terribly wrong in the management of sanitation systems in the country. Whether it be from news stories of; wastewater dumping into the Vaal, Orange and Mthatha River systems (to name but a few), raw sewage contamination of the Durban Harbour, gross ablution tender corruption in Pretoria, or from personal and interpersonal experience of contaminated water sources.

Sanitation systems are functionally one of the most valuable infrastructural sets in a country. The general purpose is to take all waterborne waste and to make it safe for discharge into a receiving aquatic environment. In order that this may be done efficaciously the entire system from the generation of some water user, through collection by the drainage network and finally processing at a wastewater treatment plant is very carefully balanced so as to maximise functionality while minimising cost of implementation. If the sensitive balance in the sanitation system is upset the result is a substandard process which exacts either a high financial toll to be corrected. If this imbalance is left uncorrected then it exacts a high physical toll on the receiving environment.

The problems which were identified for specific investigation in this research were reports of high values of pollutants in the waters around Cape Town. Evidence of repeated flow rate capacity violations for the Cape Town wastewater treatment plant fleet. Reports of disproportionately high numbers of blockages in the Cape Town drainage network.

Each of these problems comes at a significant cost. In the case of blockage numbers it is reported that the international average blockage rate is 0.3 blockages per kilometer of sewer pipe per annum (Stephenson and Barta, 2005). South Africa's average blockage rate has been estimated at 3.3 blockages per kilometer of sewer pipe per annum (Stephenson and Barta, 2005). It will be shown that a value of 12.9 blockages per kilometer of sewer pipe was experienced in 2018. Approximately four times the national average and 40 times the international average.

For the year of 2018 the city of Cape Town spent approximately R 270 million on blockage clearance alone (City of Cape Town, 2019). The national cost of blockage clearance is unrecorded so it is only possible to infer the severity of the problem from looking at case study cities where accurate records are kept.

South Africa is very water scarce country and if the national capacity to treat generated wastewater is put under stress then the result is inadequately treated wastewater being released back into fresh water resources. This comes at a significant cost to rectify or if left unattended then the consequent knock on effects are physically and financially high to any downstream community who uses contaminated water.

1.3 Motivation

A fundamental and often overlooked flaw embedded in every 'perfect' system ever designed is that these systems all have a human component to them. Whether or not this component is acknowledged is system specific. This holds true for sanitation system design where uncertainties are large, and solutions are very specific to a given problem terrain.

A specific example of this systemic flaw in engineering, which is regularly drilled into aspirant engineers in the age of technology-assisted design, is that the most important aspect of modelling is to comprehensively understand the nature of the underlying system (the so called vivirithm), before trying to create a model (algorithm) of the system.

A fundamental issue to understand with modelling is that, all models are 'wrong' since they are only a replica of the system and not the system itself. By necessity of a model being a finite approximation of a theoretically infinitely complex real-world system, assumptions have to be made and parameters have to be evaluated for significance to the model.

As mentioned earlier, design simplicity has historically been shown to maintain robustness even when the world around the designed system has changed significantly. It is the aim of this research paper to create a deeper understanding of the state of sanitation in South Africa, through the case study of the city of Cape Town.

Without a holistic and realistic understanding of the variables causing sanitation system decay in South Africa, the implementation of an effective solution is inconceivable.

1.4 Systems Thinking and Theory

There exists a field of academia called Systems Thinking and Theory (STT), proposed by Ludwig von Bertanlaffy in 1950 (Bale, 1995). STT was conceptualised as a manner in which to formulate logico-mathematical systems hierarchies describing the isomorphism of systems (Bertalanffy, 1950).

The isomorphism of systems refers to patterns of parallelism between systems from very different fields of research. In effect Laws, which hold true no matter the specific nature of the system under consideration (Bertalanffy, 1950).

An example of such a Law, is that of the exponential Law, which states that given a system with some initial number of elements, there will be an exponentially accelerating rate of element growth or decay. This is true for radioactive decay, financial interest and population growth (Bertalanffy, 1950).

To better understand the nature of STT and how it pertains to the total understanding of this research, a number of terms need to be defined.

- Entelechy – the so called 'life force' of a system, which directs its state of flux.
- Teleology – an explanation of systems regarding the purpose they serve rather than the definition of the system's causes.
- Epistemology – the theory of knowledge. How humans define, perceive, scale and validate justified belief versus opinion.
- Metacognition – an understanding of a transcendental nature, where all components of a system are considered at a high level, allowing understanding of system interrelation.
- Algorithm - a set of rules (generally logical and mathematical in nature), which is

applied in order that a physical system may be modelled in theory.

- Vivirithm - the underlying physical or 'living' system, upon which an algorithm is based.
- Heuristics – a decision making process, based on a hyper simplification of a given situation. Heuristics are used to choose the best possible option with known information; the payoff is between rapidity of finding a solution and the accuracy of the decision made.
- Homeostasis – tendency of a system to approach or maintain a level of equilibrium such that the system's elements may function.
- Living systems – systems that behave in a manor such that some form of homeostasis is maintained.
- Dead systems – systems which no longer tend towards a state of homeostasis.
- Entropy – the tendency of all things to degenerate into a state of chaos.
- Negentropy – a defining characteristic of living systems, the countering of decay through pursuit of homeostasis in order to maintain the 'life' of a system.
- Closed systems – systems wherein the entire energy, matter and knowledge remain constant. There is no escape of the system components and no entering of external components. With the exception of the universe considered as an entirety, there exist no closed systems outside of theory.
- Open systems – all real systems, wherein there is always a permeable boundary to energy, matter or information.
- Dark/opaque/black box systems – systems wherein the input and output are understood; however, the mechanisms of converting input to output are not understood.

- Transparent/white systems – systems wherein the relationship of the input and output are readily perceivable and understood.

While not all of these terms will be explicitly used, at the very least they all will appear in the background understanding of concepts discussed within the ambit of this research.

In effect, the universe is an infinite realm of information. Due to limitations of intelligence and memory, humans are only able to understand a finite amount of this infinite realm of knowledge. Epistemology is the study of the realms and processes of knowledge; which humans have generated. This implicit limitation on our collective understanding, divides the infinite realm of information, which is the universe, into two sets, that which humanity understands and that which is beyond our ken.

Even within the bounds of our world, the vast interconnectedness of systems and events is largely beyond our collective understanding. Subsequently, we are reliant upon transferring the understanding of one system onto the behaviour of another. Due to the enormous complexity of life, there is a tendency to employ heuristics to achieve a form of action to the best of our understanding.

When perceiving the entelechy of a system we can infer that other systems of a similar nature will behave in a similar fashion. Understanding the teleology of systems means that there is an ability to perceive the outcome of a system even though it may have different initial conditions to the one from which the understanding of teleology sprang.

What STT allows for is a macro-system analysis, using heuristics applied on a large scale to provide solutions, which may not be entirely perfect, but which are adequate. This is an essential concept to employ in research such as this, where there is limited information available from which, understanding of internal system mechanics may be drawn.

For example, in the case of Africa, there are many states which have each had specific starting conditions and through similar decisions (and often similar mistakes), they have all ended up with similar problems. From this it is possible to infer that due to understanding the teleology of a system, which is undergoing similar decisions and processes, there will be a similar outcome.

1.5 Methodology and Scope

The methodology of research is of great importance. With a well formulated methodology, one defines a general research path to be followed. The benefits of this general research path, are such that the researcher has a specified direction in which investigation should proceed; however, simultaneously it allows some leeway for the researcher to investigate alternative paths without losing focus of the predefined purpose of the research. The methodology of this research is presented graphically in Figure 1.1.

This research was grouped into three sections. Each section was further divided into subsections (each subsection with an accompanying chapter).

The first section, Definition, consists of an Introduction, Literature Review and Problem Conception subsections.

The Introduction was created to provide a general understanding of the path of research wherein the research would be developed. From the Introduction it was possible to decide on where the most relevant regions of understanding for this research were located.

Possibilities for specific research which were contemplated were; implementation of artificial intelligence (AI) to model blockage patterns. Investigation of possible critical flushing points to enhance blockage clearance in problematic sections of a drainage network.

Finally, the chosen realm of interest was the underlying causation of the observed blockage

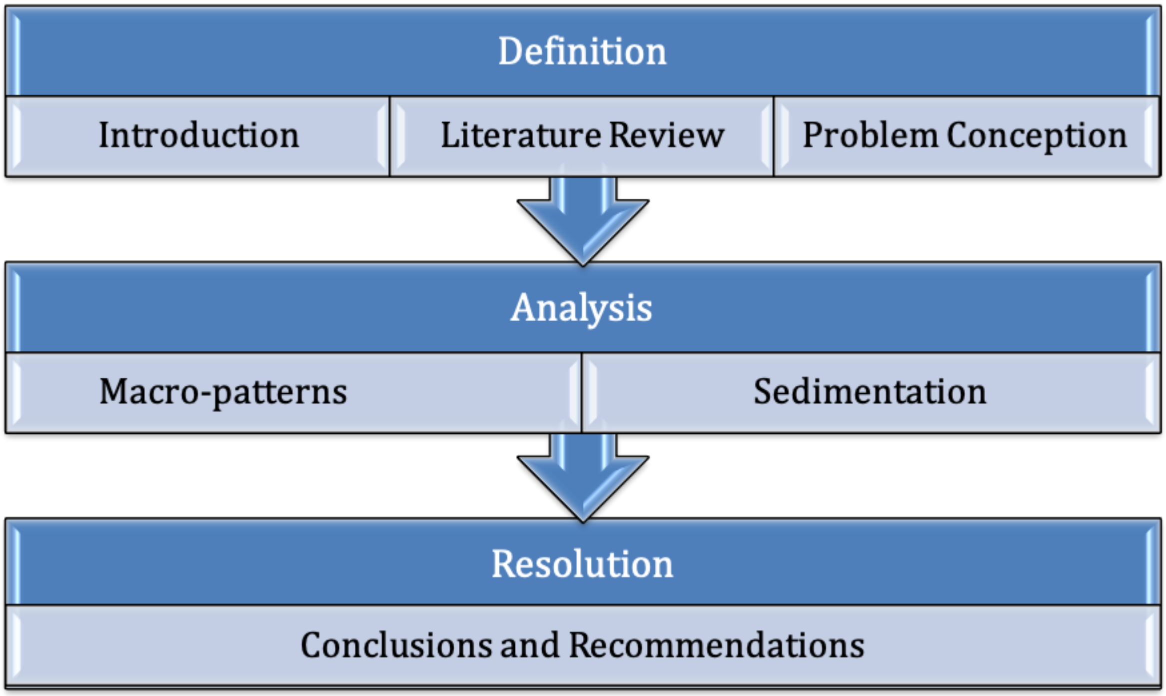

Figure 1.1 Research Methodology

patterns in combination with other patterns showcasing system fragility or decay and the causal effects of these patterns.

The Literature Review focuses on two sets of topics, namely hydraulic considerations and societal engineering considerations.

The Problem Conception subsection was created to provide an understanding of the case study in two tiers. General understanding of the system of the city of Cape Town and specific understanding of the city's sanitation system. Furthermore, it was necessary to expand on what could be extrapolated and inferred from the understanding of the showcased sanitation system patterns in Cape Town and what this meant for the rest of South Africa.

Cape Town was chosen as the case study city because it has some of the most reliable and readily available information of any of the cities in South Africa.

Two specific points of interest which led to the choice of Cape Town as the case study city were that it had the highest Greendrop Score of any city in the country. This in conjunction with an understanding that despite Johannesburg and Cape Town having similar population sizes, they have a huge difference in sewer blockage numbers. Johannesburg cleared 61 928 in 2018 (de Jager, 2019) in comparison to Cape Town's 122 706 blockages cleared in the same year. Almost exactly double the number.

A number of other important factors which reflect on the state of sanitation in South Africa but were not further explored in this research were an investigation of the age of a conduit system versus the number of blockages experienced in the system. This would indicate how the blockage patterns varied with the age of the infrastructural system. Investigating the link between land usage and wastewater type generated as well as the blockage patterns linked to certain land uses. An example of this would be an investigation of restaurant areas which often dump large volumes of cooking oil into the sewer network causing fat buildups. A final consideration was the proportion of the population using waterborne sanitation versus waterless sanitation. And how efficacious these two systems are in their areas of deployment.

The second section, Analysis, consists of Macro-patterns and Sedimentation. The purpose of this section was to provide understanding of quantified systems of decay, and an in-depth technical evaluation of one of the patterns.

The Macro-patterns subsection was based on three sets of data provided by the city of Cape Town. These three macro-patterns are; the flow-rates routed through the wastewater treatment plants, the reliance of the city's sanitation system on pumping and the blockage patterns in the sanitation system.

The Sedimentation subsection was created to showcase the theoretical impact of sedimentation patterns in the sanitation system. Furthermore, this subsection provides an evaluation of the precision, accuracy and trustworthiness of some commonly implemented sedimentation equations.

The third section, Resolution, consists solely of Conclusions and Recommendations, based on the findings of the research. The Conclusion and Recommendations subsection summarises the findings of the research as well as providing a coalescing of understanding of the implications of the research findings. Furthermore, recommendations are made for further investigation into possible solutions to the problems defined in the body of the research.

Chapter Two

Literature Review

2.1 Introduction

Nothing in the real world happens in a vacuum. This is to say that every event, no matter its nature has a series of causes and consequences, both direct and indirect. To this end the overarching purpose of a literature review is to provide contextual understanding and depth of insight into a given problem. A conceptual anti-vacuum of information describing the system in which the report was constructed.

It must be stated that in civil engineering the general considerations of a sanitation systems are oriented around hydraulic principles. The specific nature of this report is such that a macro perspective is taken of total sanitation system patterns. While an understanding of hydraulic principles is inarguably necessary, it is not the sole focus of this report. As such hydraulic principles are reviewed in the first half of this chapter to provide insight to the reader of the complex nature of sanitation system behaviour and to provide some insight into causes and effects of the macro patterns under analysis later in this report. The second half of this chapter is dedicated to providing insight into ideas more commonly found in societal engineering.

2.2 Sewerage System Components

2.2.1 Conduits

By the metric of size and cost the most significant portion of sewerage networks are comprised of conduits, accounting for some 80-90% of sanitation system costs (Bakalian *et al.*, 1994). Conduits are any channel section within a network, which transfer fluid. They may be open channel or any geometric variation of a closed section (example given rectangular brick culvert or a concrete pipe). In sewerage networks the closed section varieties of conduits are generally more common since they encapsulate the sewer flow, thereby reducing spillage and escape of noxious gases.

Conduits are generally split up into two classes, namely gravity conduits and pressurised conduits. These correspond to the two primary modes of motion induction in a sewerage network, being gravity and pressured flow (van Heerden, 2014).

Gravity conduits will effectively experience open channel flow with occasional pressurised flow induced when surcharging occurs (van Heerden, 2014). This pressurisation of gravity conduits may cause adverse effects since it places a load function on the conduit for which it was not designed. This may lead to pipe damage or rupturing in extreme cases.

Pressurised conduits are designed to operate under pressure. The pressure is generally supplied by a pump system placed at the downstream end of the pressurised conduit.

Conduits are grouped according to their diametric size. From smallest to largest these groupings are; reticulation, collector (trunk and rising mains) and bulk. A schematic layout of sewerage system components is provided in Figure 2.1

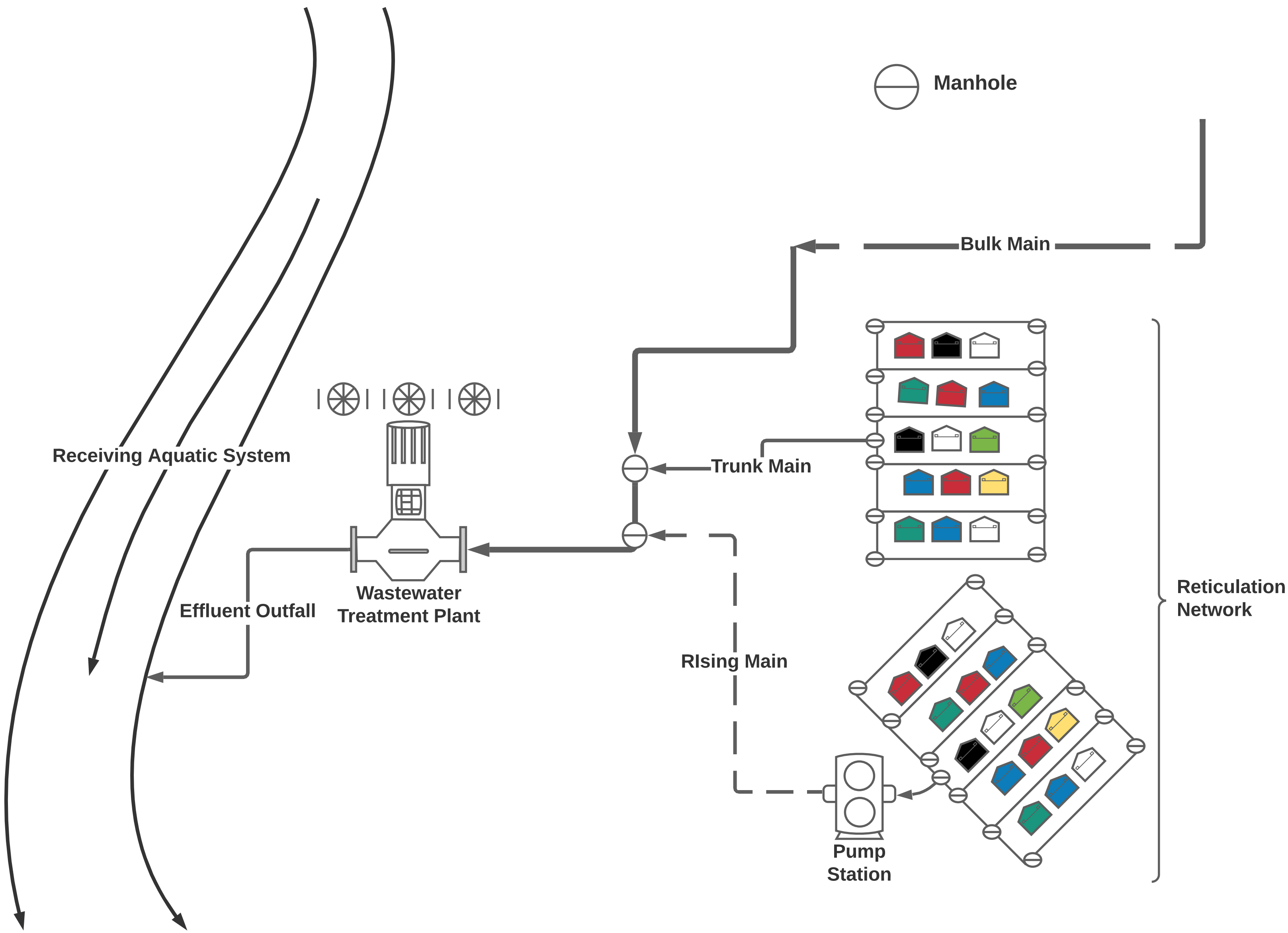

Figure 2.1 Wastewater System Component Diagram

2.2.2 Wastewater Treatment Plants

Wastewater treatment plants by consideration of process are one of the most important parts of a sewerage network as they perform the function of reducing influent toxicity to an acceptable standard in order that the resultant effluent may be safely discharged into the receiving environment (Department of Human Settlements, 2019). There exist three main classes of wastewater treatment plant systems.

- Conventional (large volumes of wastewater)
- Pond (small volumes of wastewater. Generally implemented in rural areas)
- Package purification (generally small volumes associated with small communities)

Conventional plants consist of two further subgroups, namely, bio-filtration and activated-sludge plants. Bio-filtration plants are implemented when a lower quality of effluent is required; whereas, activated sludge plants are implemented when a higher standard of control and subsequent effluent quality is required (Department of Human Settlements, 2019).

Pond systems are reliant on sunlight and algal blooms to treat the wastewater. As such the process is relatively slow and requires significant space. Hence the general use in rural areas (Department of Human Settlements, 2019).

Package purification plants are generally compact, technologically advanced and modular in design. Many system components are prefabricated offsite and then assembled on site. Package purification plants may be modelled on conventional wastewater treatment plant design; however they tend to use mechanical and chemical processes to accelerate wastewater treatment (Department of Human Settlements, 2019).

2.2.3 Manholes

Manholes are an essential component of sewerage network maintenance since they provide access into the network for observation and maintenance (Van Zyl and Van Dijk, 2011).

Furthermore manholes can be designed in such a way as to provide a measure of flow attenuation and hydraulic control (van Heerden, 2014).

Design considerations of manholes should also account for a minimisation of flow impedance and infiltration and maximisation of durability (Van Zyl and Van Dijk, 2011).

The placement of manholes should be spaced at either 100 or 150 m intervals, depending on whether the local operative body has either hand maintenance means or mechanical maintenance means. Furthermore manholes should be included in the event of the following circumstances (Van Zyl and Van Dijk, 2011):

- Changes in grade or direction
- Positioning on steep grades (1:10 or greater)
- Upstream end of any section serving 3 or more dwellings and have an associated section of greater than 50 m
- At the intersection of sewer lines and roads
- At all sewer junctions

2.2.4 Drop and Diversion Structures

Drop structures are a commonly used form of hydraulic energy dissipater. They are generally found in sewerage networks constructed with steeper natural topographies.

The purpose of a drop structure is to dissipate energy thereby preventing such unwanted phenomena as surcharging or scour (van Heerden, 2014). Drop structure numbers should be limited though since they cause the formation of a hydraulic jump which in turn releases increased amounts of corrosive substances, such as hydrogen sulphide and can cause conduit degradation (Ebtehaj and Bonakdari, 2013).

Diversion structures such as weirs are used to route flows. This allows for the control of network flow and pollutant loading. Effective control of network loading should prevent adverse effects such as flooding and surcharging during high flow periods (van Heerden, 2014).

2.2.5 Pumps

Pumps are any form of mechanical pressurising unit, which causes the pressurisation of flow thereby allowing for the overcoming of frictional resistance and adverse slopes.

In flatland regions the gravity conduits will generally be buried with increasing cover. This allows for the maintenance of a minimum slope. At the end-points of these gravity conduit systems it is then necessary to re-elevate the flow either as wastewater treatment plant influent or as a load into a neighbouring drainage region (van Heerden, 2014).

2.3 Sewage Flow

Sewage flow is a complex hydraulic system due to its high level of variation both temporally and spatially. Sewage flow is generally classified as turbulent, unsteady and nonuniform (Mays, 2001a).

Furthermore, good design practice should take into consideration the generation and release of hydrogen sulphide (H_2S) gas, which combines with oxygen to create sulphuric acid (H_2SO_4). Sulphuric acid is highly corrosive to concrete, which is a key material in conduit construction.

Hydrogen sulphide generation is largely caused by biological activity in the effluent. The most important considerations for hydrogen sulphide generation are (PIPES, 2009):

- Sewer retention time
- Velocities that are not self cleansing
- Sedimentation
- Temperature
- Biochemical oxygen demand (BOD)
- Dissolved oxygen (DO)
- Dissolved sulphides (DS)
- Effluent PH

With increasing concentrations of dissolved hydrogen sulphide, flow velocities and turbulence, there is a corresponding increase in release of hydrogen sulphide into the unfilled area of the pipe cross section. Subsequently there is a concomitant increase in sulphuric acid accumulation on the conduit walls leading to increased corrosion (Van Zyl and Van Dijk, 2011).

Consequently the limitation of flow velocity tends to cause subcritical flow in sewers (Mays, 2001a).

There are 3 regions of flow in a sewer. Given below in Figures 2.2, 2.3 and 2.4, respectively representing the different flow conditions for the entrance, exit and conduit sections.

The four cases in Figure 2.2 respectively represent (Mays, 2001a):

I Nonsubmerged entrance, subcritical flow

II Nonsubmerged entrance, supercritical flow

III Submerged entrance, air pocket

IV Submerged entrance, water pocket

The four cases in Figure 2.3 respectively represent (Mays, 2001a):

A Nonsubmerged, free fall

B Nonsubmerged, continuous

C Nonsubmerged, hydraulic jump

D Submerged

In the following figures "y" denotes the critical depth of flow.

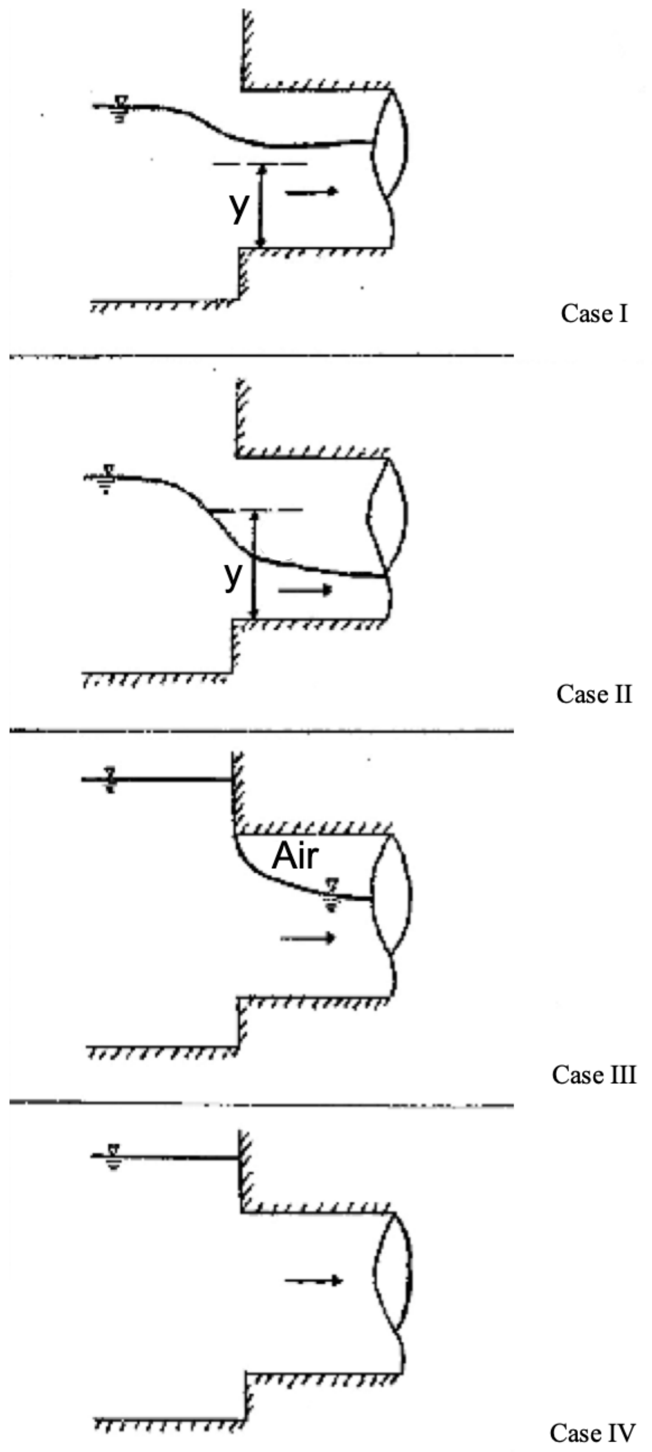

Figure 2.2 Conduit Entrance Flow Conditions Amended from (Yen, 1986)

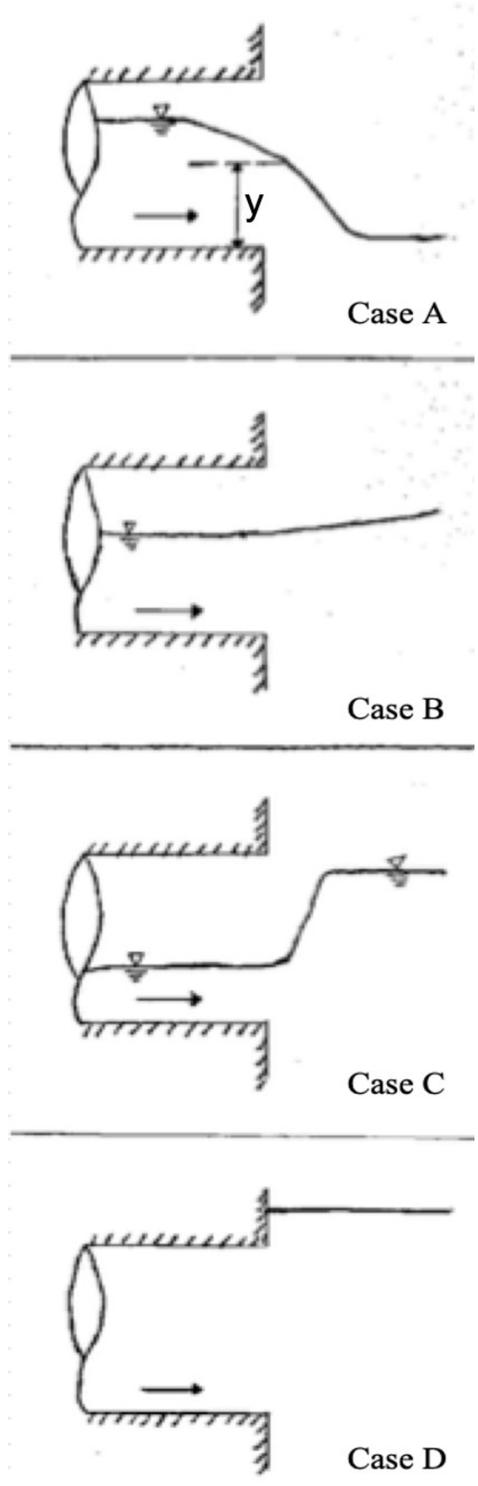

Figure 2.3 Conduit Exit Flow Conditions Amended from (Yen, 1986)

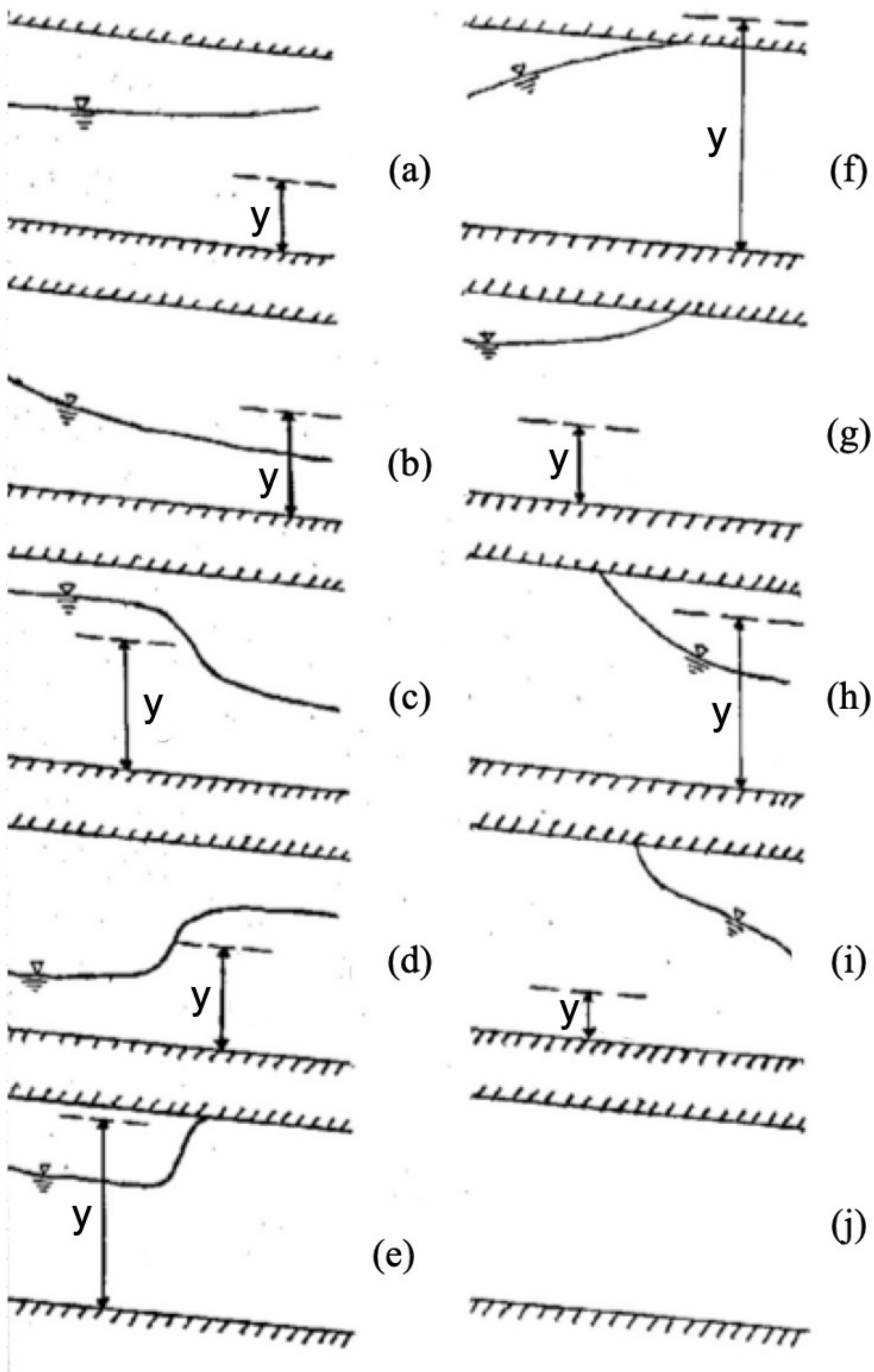

Figure 2.4 Conduit Midsection Flow Conditions Amended from (Yen, 1986)

The 10 cases in Figure 2.4 respectively represent (Mays, 2001a)

(a) Subcritical

(b) Supercritical

(c) Subcritical to supercritical

(d) Supercritical to subcritical

(e) Supercritical jump to surcharge

(f) Supercritical to surcharge

(g) Subcritical to surcharge

(h) Surcharge to supercritical

(i) Surcharge to subcritical

(j) Surcharge

Sewage flow is highly complex and variable. The nature of flow will vary in accordance with flow rate and conduit cross-section. In effect the nature of flow will be temporally linked to the hydrograph of a conduit section and spatially linked to the conduit cross sectional variation.

The flow conditions shown in Figures 2.2, 2.3 and 2.4 are all regularly experienced in a sewerage network. The flow conditions shown in Figures 2.2 and 2.3 would likely be found at conduit junctions (manholes) therefore these flow conditions are more likely to be representative of spatial variations in the sewerage network. Figure 2.4 shows flow condition variation more likely to occur with temporal variation of flow volumes and energies.

Sewer systems constantly experience variation of flow rates (due to user demand variations and storm water ingress) and that these flow rate variations cause a variation of velocities and flow energies at a given point in the sewer network. What is important about understanding flow in sewers is that with a variation in velocity and flow energy there is a variation in the ability of a specific section of sewer to maintain self-cleansing velocities.

In Section 3.3 Topography and Geology it will be shown that the city of Cape Town has serious sedimentation intrusion problems. These lead to a reduction in flow area and an increase in blockage probability.

The ability of a sewer to maintain a self-cleansing velocity is linked to the flow conditions at a given point in time. If a sewer experiences adverse flow conditions which cause increased settling out of sediments, then at or near this location a sediment deposit will form. This deposit will in turn affect the flow conditions at the point it is formed and possibly lead to issues of greater solid matter trapping and consequent blockages.

While flow conditions are not investigated in depth in this research a basic understanding of flow conditions is essential to understanding later sections of this research with regard to sedimentation and its concomitant effects on blockage patterns.

From the information supplied by GLS Consulting, it is estimated that the sewerage network of Cape Town cumulatively has a length of 9 539 km. Therefore there should approximately be between 64 000 - 95 000 manholes. At each of these manholes there will be some form of flow condition variation. One begins to understand the turbulence, unsteadiness and nonuniformity of flow in a sewerage network and the consequent variations in behaviours of phenomena of flow velocities, corrosive gas release, sedimentation and frictional and local losses.

2.4 Conventional versus Simplified Sewerage

Globally it is recognised that one of the biggest constraints placed on civil engineering project implementation is cost efficiency. This constraint is even more pronounced in South Africa where there is rampant corruption and a relatively weak currency. Consequently, a brief overview of conventional versus simplified sewerage design is supplied.

Conventional sewerage has been in existence in its current format since roughly 1840 in Hamburg, Germany (Van Zyl and Van Dijk, 2011). It is reliant on a system of conduits built to operate at full capacity during high flow periods with some capacity allowance being made for storm water ingress. Further design constraints are applied to minimum velocities of approximately 0.7 m/s (Van Zyl and Van Dijk, 2011). The purpose of these minimum design constraints is to cater for the transport, erosion and deposition of sediment.

The United Nations Development Programme and the World Bank funded simplified sewerage design. The project to prototype simplified sewerage design was undertaken in Brazil in the 1980s with an international team of researchers tasked with creating a new design system to reduce the costs of sewerage drainage networks, since the cost of conventional sewerage was providing a financial impasse on adequate sanitation services in the favelas (Brazilian townships). This was in large part due to an unprecedented population explosion and urban densification (Bakalian *et al.*, 1994).

Where conventional sewerage design is reliant on minimum flow velocities, simplified sewerage design was created with the defining characteristic being a minimum tractive force in the conduits and a diameter specific minimum slope. This allowed for relaxation of design standards and effective cost savings being made on reduced pipe diameters, reduced labour costs through reduction in pipe slopes and consequently reduced excavation costs. Furthermore, savings were made with the minimisation of the requisite number of manholes (a relatively very expensive drainage network appurtenance) (Bakalian *et al.*, 1994; Van Zyl and Van Dijk, 2011).

The net effect of these design constraint relaxations was that simplified sewerage design implementation leads to a reduction of construction cost of between 20 and 50% compared to conventional sewerage design (Bakalian *et al.*, 1994; Van Zyl and Van Dijk, 2011).

Of further interest is that in Sao Paolo it was estimated that simplified sewerage design accounted for 75 blockages per 1 000 km of sewers each month (relative to the 1980s) (Bakalian *et al.*, 1994).

simplified sewerage design was initially designed for the poorest regions of Brazil; however, it proved so successful and so much more cost efficient that it was implemented right across Brazil and a number of other Latin American countries, as well as into certain states of the United States of America and Australia (Bakalian *et al.*, 1994).

Since the focus of this research is based on investigating the state of sanitation in South Africa, the implementation of simplified sewerage design in South Africa was not considered in depth. However, considering the strong similarities in nature of Brazil and South Africa as two rising third world countries there is a strong case to be made that in future research simplified sewerage design implementation in South Africa should be investigated further.

2.5 Risk versus Uncertainty

Risk is the quantification of likely effect through probabilistic analysis of generally known system variables (CSIR, 2017a). Uncertainty on the other hand pertains to everything which is unknown about the future of a system (Thoma, 2010).

Uncertainty may be viewed as the parent class of Risk. This is to say that a risk assessment cannot be conducted in the absence of any probabilistically calculable outcomes. Where risk assessments are possible and generally sought after when systems are relatively well understood, and future system behaviour can be predicted with a suitable degree of certainty, the same cannot be said of future uncertain systems. In such uncertain systems, risk assessments becomes valueless and an uncertainty oriented evaluation process is advised (Haldane, 2012).

What has historically been observed of uncertainty-governed systems is that simplicity reigns supreme (Haldane, 2012). Application of simplicity-oriented design allows for the construction of robust systems, which are most likely to survive in times of uncertainty.

2.6 Data Quality

2.6.1 Bias

The current state of South Africa is one of great uncertainty. Judging by such indicators as ratings agency downgrades, levels of service delivery dissatisfaction and alleged corruption the country as a whole is in a downward spiral. The severity of the decline is hard to quantify and as such it is generally open to conjecture and debate. This leads to a serious issue of data quality emerging from the systems of South Africa.

There exists a national framework for the regulation and evaluation of water and wastewater service providers titled the Blue Drop and Green Drop reports, respectively. The purpose of these reports is to give an annual rating of the level of service, which is provided by municipal functionaries. The awarding of Green Drop certification shows that the wastewater system is functioning in a healthy manner. The awarding of a Purple Drop certification shows that the wastewater system is seriously underperforming.

When reviewing the structure of the Green Drop certification one cannot help, but to notice the similarities between a "world class system" of wastewater treatment standards and the standards of the South African education system. Both touted by politicians as adequate tending towards exceptionally capable while in reality both are failing the people of the country.

The last Green Drop report was published in 2013 (Ntombela et al., 2016). By the numbers, the City of Cape Town Metropolitan Municipality had the highest performance rating of any municipality accounting for 11 of the 40 awarded Green Drop statuses in 2011 (Department of Water and Sanitation, 2011). While stated that the reports should be readily available to the public, these are not easily found in a complete format.

Another benchmark of statistical standards is the annually published General Household Survey produced by Statistics South Africa. This is supposed to be a document, which clearly quantifies the state of South Africa's living conditions and other topics of interest. The survey accords 7 pages of information to Water, 5 pages to Sanitation and 15 pages to Education (Statistics South Africa, 2018). While it is fully acknowledged that education is an extremely important topic in South Africa it is questionable that it holds import greater than the combination of both water and sanitation. Furthermore, the survey has a number of inherent mistakes, such as in the Health appendices, the row dealing with malaria states that there are no recorded values for female cases of malaria in contrast to 6 000 reported cases of male infection. The row then goes on to summarise the total number of malaria cases as 8 000. It must be noted that the footnote of the table states that, due to rounding, the numbers do not necessarily add up to total. However the question stands, what is the trustworthiness of data, which breaks international rounding standards by going from 6 000 to 8 000 on the grounds of rounding (Statistics South Africa, 2018)?

While this is only one example of questionable data handling, another would be the use of statistical tricks like the addition of the word "improved" before "sanitation". What are the bounds of this improvement? In statistics, use of words such as these have a similar effect to toothpaste advertisements stating that use of their product will result in 100% whiter teeth. There is no benchmark mentioned against which "improved" may be measured. The result is deception.

There exists a concept of academic confirmation bias. The basis of this bias is that academic research is only funded if it aligns to the interest of a munificently flush or politically powerful entity. This indirectly excludes the research of unpopular topics. This academic confirmation bias plays a significant role in the direction of academic pursuit the world over. In South Africa it has the effect of generally placing high quality data and research behind pay walls.

The news is not generally considered a suitable academic resource; however, in a quality data starved environment the news can be used as an indicator of events happening. There exists a news ombudsman whose prerogative it is to ensure that all news is fair and impartial (ethically reported). When coupling this with the relatively uninteresting nature of stories on the flushing of untreated wastewater into the Vaal river system, the Durban Harbour and the Cape Town river system, it can be argued that news stories may be used in the same way that *E. Coli* is used to benchmark water quality. *E. Coli* is termed an indicator organism, which is to say that generally where there is *E. Coli* there are also other unpleasant pathogens. In this line of argument, as long as clarification is provided that the origin of information is indeed a news publication and that the story is published in more than one publication, it is possible to justify using news as an indicator observation. There is no statistics ombudsman in South Africa, well at least none that is publicly known to function in a fair and impartial manner.

Arguably there exist two kinds of data in South Africa. 'Smart' data as is showcased by the Blue and Green Drop reports and the General Household Survey, where statisticians and some other functionaries have undertaken to organise and represent data in a manner, which aligns to the interests of an entity intent on using carefully selected data to represent a form of the ‘truth’ while obfuscating the whole Truth.

In contrast there is ‘dumb’ data. ‘Dumb’ data is showcased in the recordings of such events as rainfall, sewer flowrates or sewer blockages. ‘Dumb’ data is in large part the basis of big data, where so many data generating events occur and the patterns are so large that, until data processing reached a suitable level of efficiency in the recent past, there was very little inclination to order ‘dumb’ data. Now, however, ‘dumb’ data is qualitatively one of the only reliable sources of information in South Africa.

‘Dumb’ data stems from the recording of events happening that, generally very few people care about. ‘Dumb’ data under any name is generally considered essential to the work of a lot of science. Shown below in Figures 2.5 and 2.6 are a series of images showing the ’health’ of some of South Africa’s most essential dumb data collectors. Namely those of stream flow gauges and rainfall gauges.

In both graphics, it can be clearly seen that in the past two decades there has been a steep decline in numbers of stream flow and rainfall gauges in South Africa. While this may be of little concern to the average citizen of the country, these gauges serve as the ‘eyes and ears’ for the hydrological community of the country. The information gathered from these gauges is used to calibrate and run our hydrological models, which are some of the best structured, yet most complicated models used in the country. The value of these models lies in the ability to accurately predict the likelihood of rainfall events and where the water will end up. Thereby, allowing for the planning of water resource management.

This is one of the oldest functional ‘dumb’ data systems in the country. The results of it are vital to the well-being of the entire population and yet these ‘eyes and ears’ are systematically being blinded.

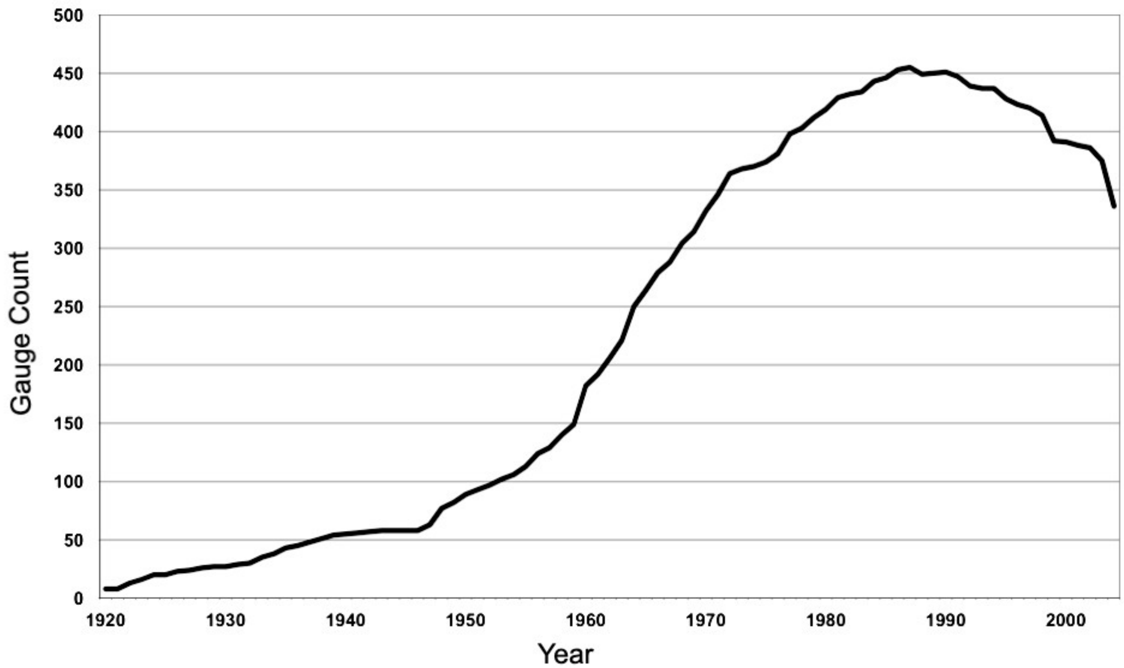

Figure 2.5 Stream Flow Gauge Count in South Africa (Pitman, 2011)

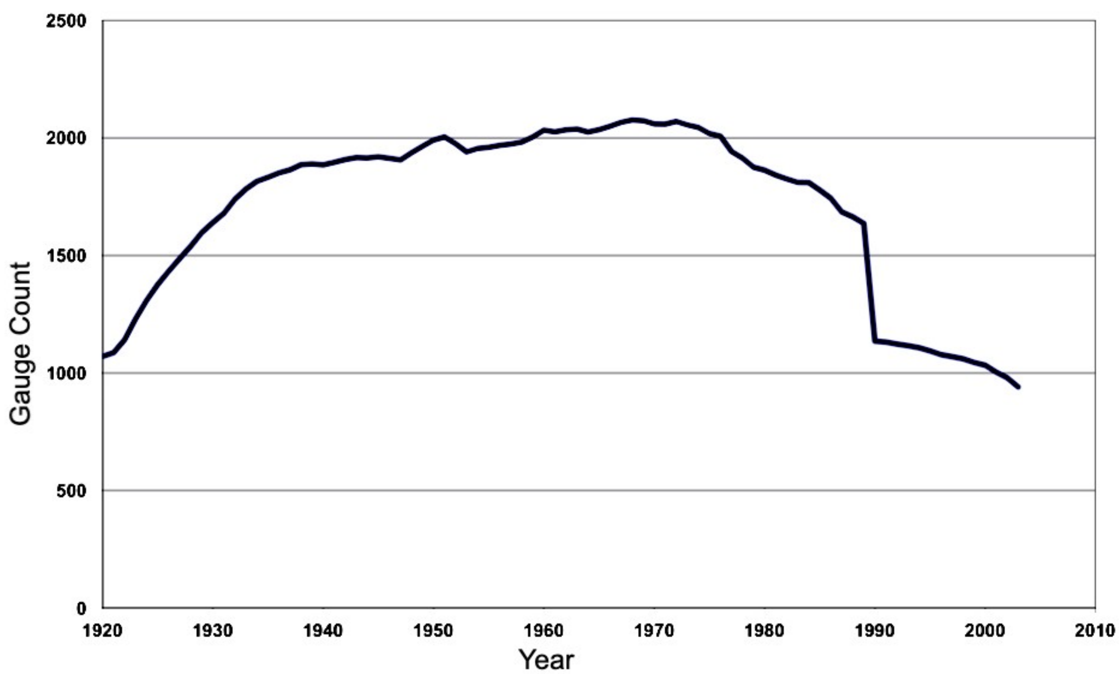

Figure 2.6 Rainfall Gauge Count in South Africa (Pitman, 2011)

While these are only two graphs of two sources of 'dumb' data, the patterns of decay reflected therein can be extrapolated to most of South Africa's data collection resources with a reasonable degree of certainty.

Data quality and its validation have become of paramount importance in South Africa due to the decline in data sourcing and harvesting processes. Any reasonable conclusions have to be drawn from well-validated data-sets; otherwise the foundation of any conclusions comes into question.

2.6.2 CRISP-DM Methodology (Wirth and Hipp, 2000)

Data mining is a relatively new concept to the realm of science. Its purpose is to take vast quantities of data (what can be comparatively seen as white noise information) and extract specific subsets of the data (key note information).

Initially when data mining began to gain serious traction in the realm of science, it did so in an uncoordinated fashion. Subsequently there was need for a standard process to be followed to make data mining a more universally accepted and understood medium of scientific investigation.

What was developed is known as the CRoss Industry Standard Process for Data Mining (CRISP-DM) Methodology.

The CRISP-DM Methodology has six phases which are iteratively looped through until a suitable end point of data understanding and functionality is achieved. These six phases and a brief explanation of each phase is given below.

- Business Understanding - what is being sought in a given project (business) with respect to what the data has to offer.

- Data Understanding - What does the data have to offer in terms of extractable information.
- Data Preparation - the conversion of raw data into ordered data. This phase can involve numerous steps of cleaning, ordering and filling data depending on the source quality. This is arguably the most time consuming and important phase in order to guarantee sound modelling.
- Modelling - Models are selected, applied and calibrated against test sets of data.
- Calibration and Verification - The models efficacy and accuracy are tested against the goals laid out in the Business Understanding phase. The models are consequently either continued, altered and retested or discarded entirely.
- Deployment - the information mined out of the data is presented to interested parties. It is essential that in the deployment phase the findings of the data mining process are readily digestible to the interested parties.

Each of these phases must be looped through until a suitable outcome is achieved. The looping can be between neighbouring phases or through the entire method. Each different project will have different requirements and so this methodology is proposed as a guideline more than a definitive rulebook. Ultimately the power of the outcomes of the project are dictated by the skill and understanding of the data miner.

The CRISP-DM serves as a powerful tool in assessing data in a country such as South Africa where it has been shown there are probable large faults in many essential data-sets due either to human interference or through impaired data collection technology.

Chapter Three

Problem Conception

3.1 Water Resource Health

It has been mentioned previously in this report that prevention beats the cure. This sentiment is expressed in the World Health Organization – Guidelines for Drinking-water Quality (World Health Organization, 2017), albeit with different wording. This guideline provides direction and understanding for the maintenance of water quality in all countries.

Section 4.1.1 Pollutant Counts is dedicated to the chronicling of water pollutants (both biological and abiological) found in the waters of Cape Town's hydrosphere and the effects they have on all life forms and humans in particular. In short, this report clarifies the obvious, if water is mistreated then the effects are a severe decline in societal health.

Figure 3.1 is of the outflow point of the Lake of Luzern. It is the definition of fairy-tale city. To walk on the Kapellbrücke is to take a walk through a beautiful history. However, the Lake of Luzern was not always so blue and beautiful. A few decades ago there was fertiliser pollution of the water leading to eutrophication and a shade of foul green. Switzerland has one of the highest GDPs in the world and one of the most powerful social contracts. So the

Figure 3.1 Lake Luzern Overflow, Switzerland

citizens of Luzern organised funding and restored the lake to a healthy state of blue and beautiful.

South Africa has ‘not the highest’ GDP in the world and a very weak social contract. Seemingly this results in its ‘lake’ waters being various shades of not ‘blue and beautiful’ (Figure 3.2) with a tinge of scum around the borders (Figure 3.3). Instead of organising a restoration of the waters of this ’lake’ the local authorities deemed it suitable to place a ‘warning of death’ sign (in four different languages) on the shores of Zoo Lake (Figure 3.4).

It can be argued that it is very unfair to compare the Lake of Luzern to Zoo Lake because of their vastly different environments. Most notably Zoo Lake is fed primarily from storm water run off of a city. Whereas, The Lake of Luzern is fed with Alpine Gletscherwasser (glacier water).

Figure 3.2 Zoo Lake Weir, Johannesburg

Figure 3.3 Zoo Lake Scum

Figure 3.4 Poisoned Water Sign, Zoo Lake

So to provide a better counterpoint the Eersterivier is used. The Eersterivier rises from one of the highest rainfall catchments in the country and near its source it is of suitable quality (Figure 3.5) that it has sustained the relatively healthy life of a 50-year-old homeless man named Piet. This is the kind of quality found in the Lake of Luzern. However, a short distance downstream of Piet-se-Paleis (the public spaces of Stellenbosch) the Eersterivier becomes so polluted (Figure 3.6) that it would be a better qualification of the channel were it described as an open sewer rather than a natural watercourse.

The Eersterivier flows from the Jonkershoek Valley through Stellenbosch, past a number of townships and joins the Kuilsrivier next to the Zandvliet wastewater treatment plant. Finally it enters the ocean next to the Macassar wastewater treatment plant. Along this path it goes from clean water to unclean water. The only sources of this contamination are human activities, be they waste dumping and littering, fouled runoff or inadequately processed wastewater treatment plant effluent.

Figure 3.5 Unpolluted Eersterivier

Figure 3.6 Polluted Eersterivier

3.2 Case Study Introduction - Cape Town

The city of Cape Town has one of the fastest growing populations in South Africa. It also has some of the highest disparities of wealth, with the areas abutting the mountains (such as the Atlantic Seaboard and the Southern Suburbs) being home to some of South Africa's wealthiest citizens and the flat land areas (such as the Cape Flats) being home to some of South Africa's poorest citizens (du Toit, 2019).

Yet the wastewater system for the city must serve all of the citizens be they rich or poor. The economic disparities of the citizens are reflected in the underlying wastewater system; however, these are not clearly understood nor recognised.

The purpose of the following chapter is to provide a conceptual understanding of the city of Cape Town and the layout of its wastewater system.

It must be noted that by virtue of the vast majority of the wastewater system existing below ground it is categorically a 'black-box' or 'dark' system. The majority of the information presented in this chapter was extracted from the City of Cape Town's Masterplan as it is represented in GLS Consulting's proprietary GIS sewer software, Sewsan. It should be noted that a significant portion of the information in this Masterplan of the city was explicitly imposed and therefore it is not necessarily a perfectly accurate representation of the underlying system, but rather the best available knowledge base for a work such as this report. This necessitous imposition of data is a defining characteristic of 'dark' system analyses.

Since it is considered of paramount importance to have a clear conceptual understanding of the entire system relative to the geography of the city, numerous of the following graphics will be displayed with an imposed cartographic backdrop (taken from Sewsan's StreetMap functionality). All images are displayed with North in the direction of the page zenith.

3.3 Topography and Geology

Cape Town is rather interesting. The Cape peninsula mountains are separated from their neighbouring mountain ranges by a large sediment bed. In Figure 3.7 it can be seen that the dark green regions are the more mountainous and less populous areas of Cape Town, while the light green areas are the flat-lands (hence the name Cape Flats) with low variation in elevation and a high population density.

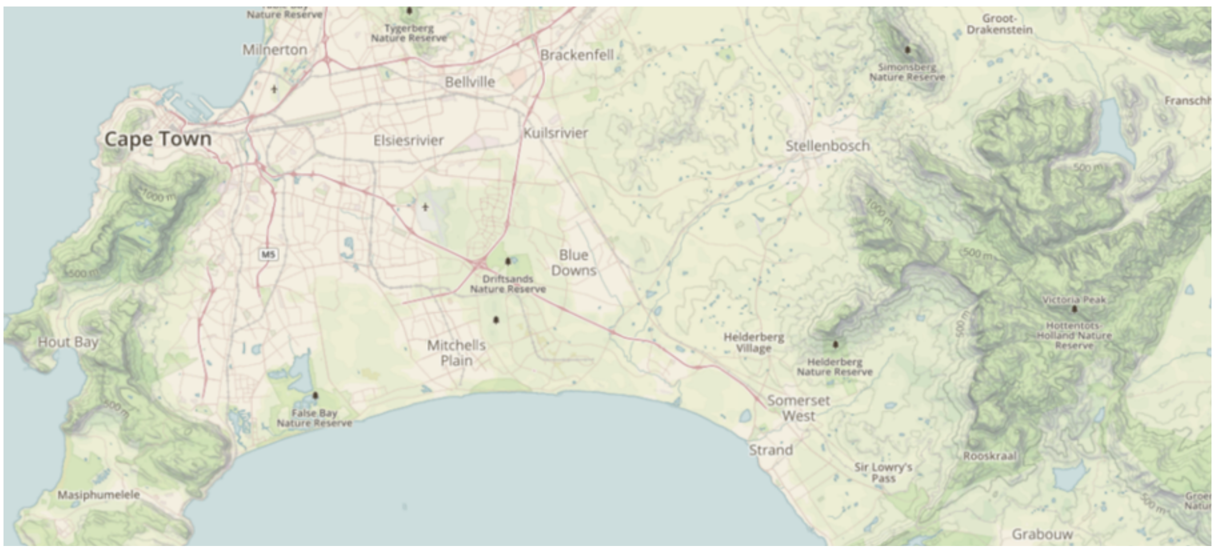

Figure 3.7 Cape Town Topographic Map (GLS Consulting - Sewsan, 2019)

Intuitively this would lead to the assumption that the populous mountainous regions would have better sanitation systems, with good slopes and minimal sediment intrusion. In Figure 3.8 it can be clearly seen that the vast majority of the city spread exists on the flat-land region walled in by mountains.

All sewer systems are prone to sedimentation. However, sediment should not be able to find its way into sewer systems. In South Africa it is a common sight to see manhole lids missing or damaged (Figure 3.10). This in conjunction with other imperfections (pipe cracking or disjointing due to ground movement) lead to sediment intrusion. When there are strong

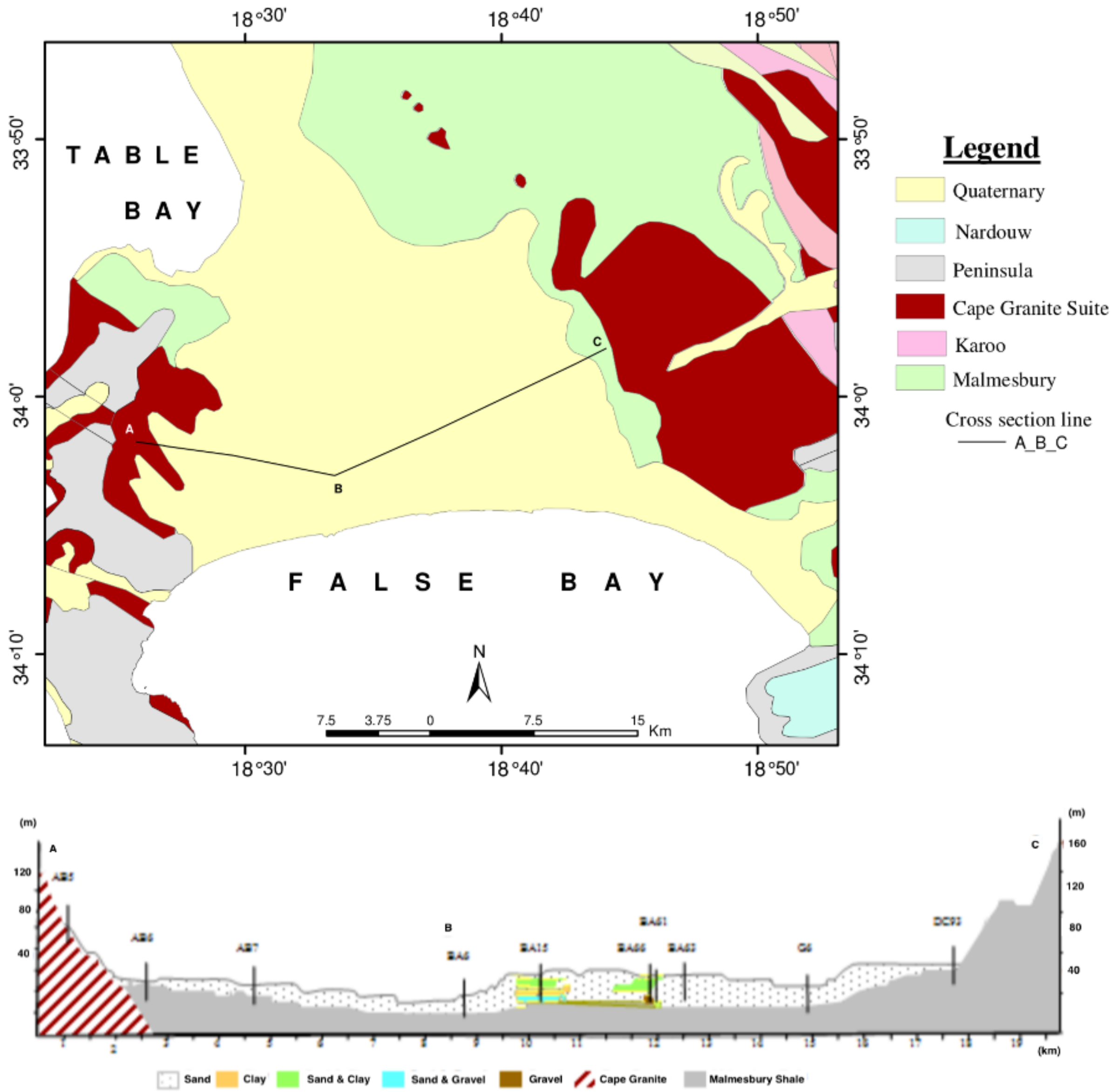

Figure 3.8 Cape Town Geological Map and Cross Section (Adelana, Xu and Vrbka, 2006)

Figure 3.9 Open Manhole

rains leading to flooding, the absence of a proper seal on manholes allows for large amounts of runoff to enter the sewer system.

In cities such as Cape Town where there is a high water table a large proportion of the drainage network will lie in the wetzone thereby allowing intrusion of sediment carried by infiltrating groundwater. Furthermore, in many areas illegal domestic connections are made either for storm water or sullage drainage. This allows for surface runoff (with relatively high loads of sediment) to enter into the wastewater system.

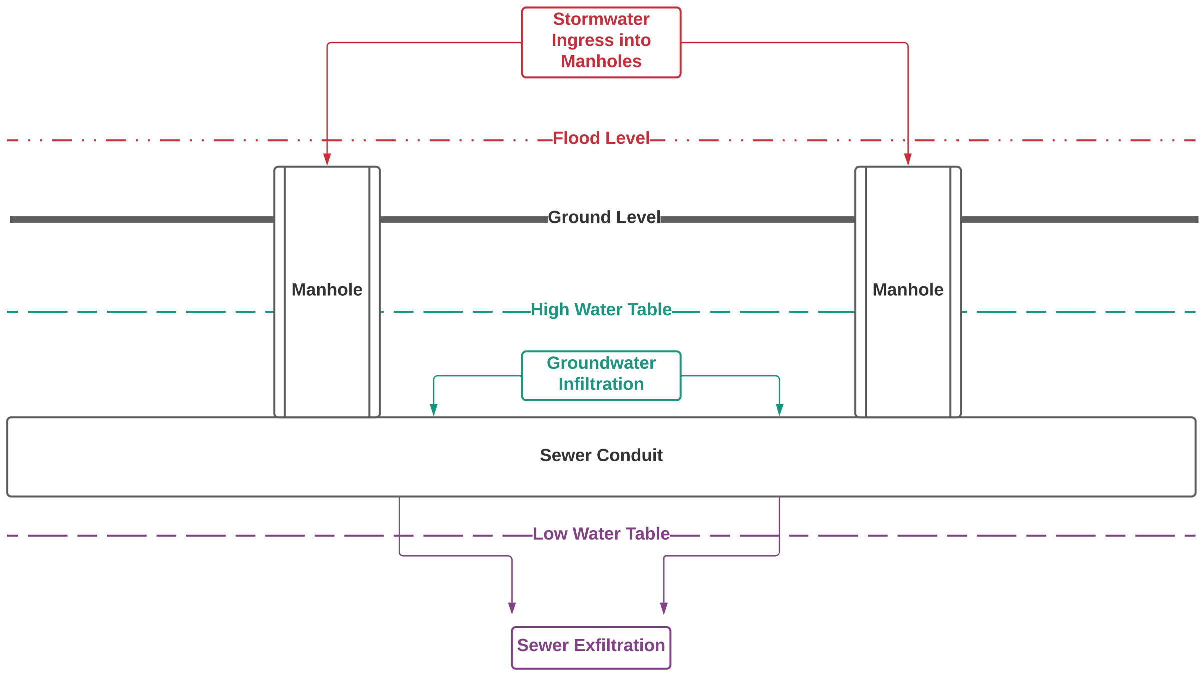

Figure 3.10 Sediment Intrusion into Drainage Network

The City of Cape Town was asked for information on the sandtraps in the wastewater system. Only two of four regions replied with information. However, this information is still valuable. In these two regions there are some 77 sand traps. Most are cleaned on a monthly cycle while the more problematic ones are cleaned on a weekly cycle.

The collective amount of sediment removed from these sandtraps is approximately 678 tons per month. Meaning that in a year some 8 136 tons of sand are removed from these two regions' drainage networks. This is clear evidence of a high sediment intrusion problem.

3.4 Drainage Systems

The wastewater system of Cape Town is one of the most valuable infrastructural assets of the city. According to GLS Consulting, the construction value for the wastewater system totals R 36 billion. The subtotals for each specific drainage area is given in 3.1. Collectively this system transports the waterborne waste for an estimated 4 524 111 people (du Toit, 2019).

Given in Figure 3.11 is a breakdown of the costs of the subsystems. The collective cost of the gravity and rising main networks is 84%, while the wastewater treatment plants collectively account for 14% of the construction cost. The areal layout of the drainage systems is provided in Figure 3.12. The specific construction costs of structures relative to the drainage networks are summarised in Table 3.1.

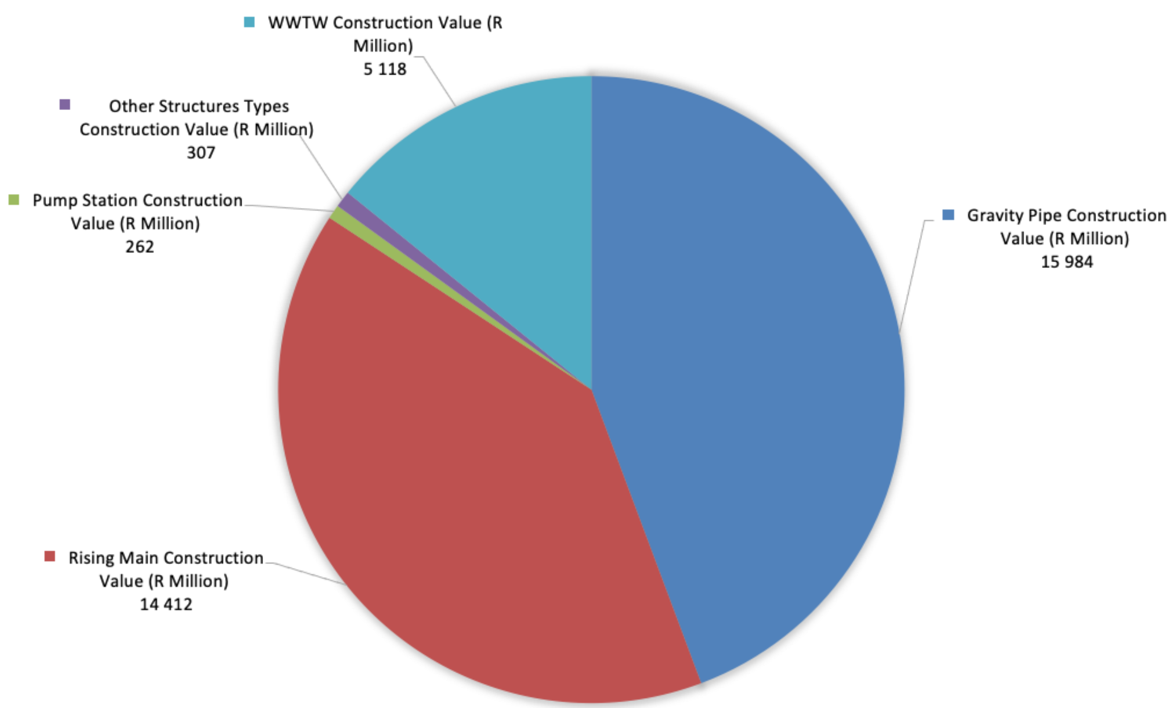

Figure 3.11 Cape Town Wastewater Network Drainage Systems (GLS Consulting, 2019)

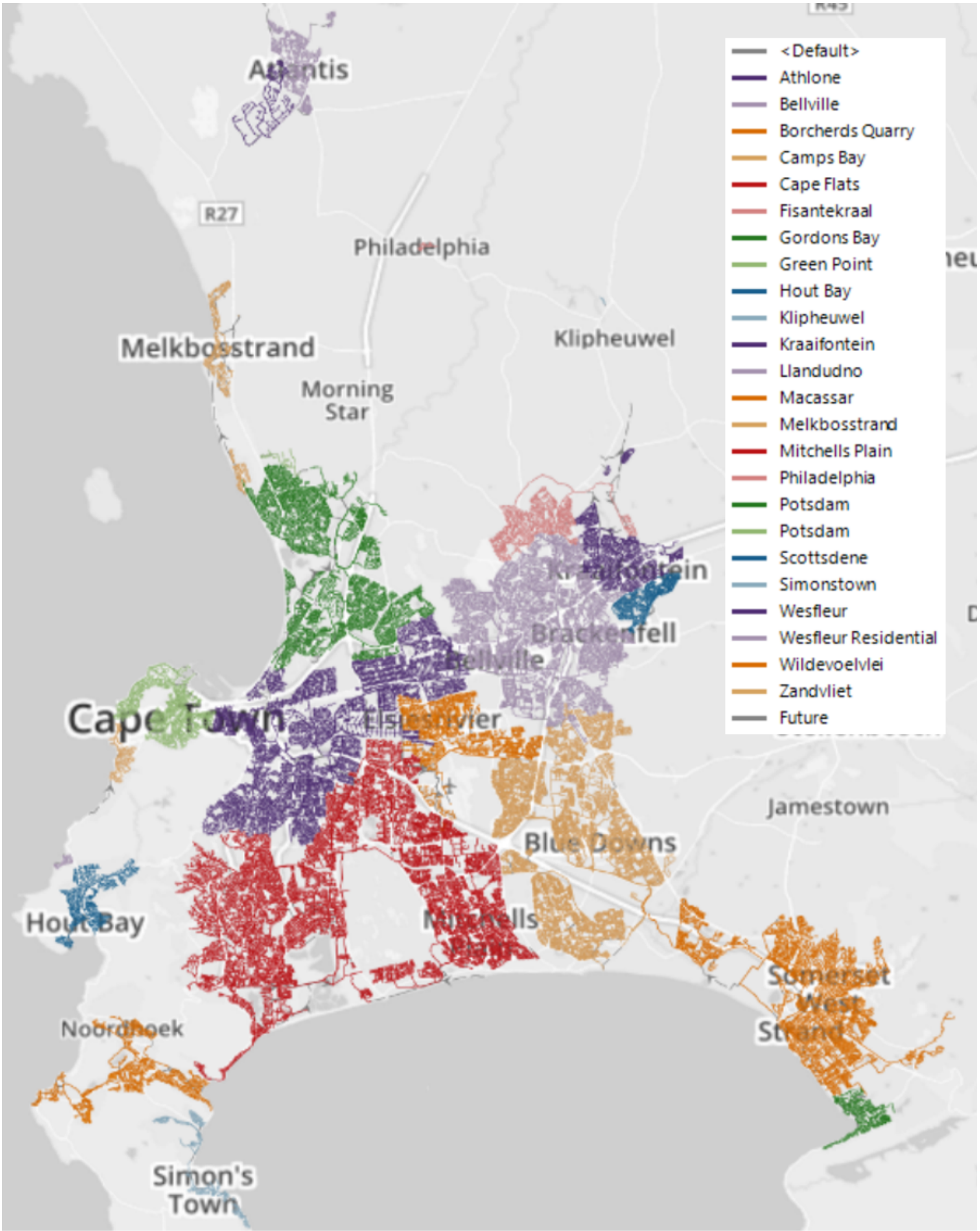

Figure 3.12 Cape Town Wastewater Network Drainage Systems (GLS Consulting, 2019)

Table 3.1 Summary of Cape Town Wastewater System by Drainage Area (GLS Consulting, 2019)

Existing Drainage Area	Drainage Network Construction Value (R Million)	Pump Station Construction Value (R Million)	WWTP Construction Value (R Million)
Rising Main	14 412.0		
Athlone	2 145.4	26.8	605.2
Bellville	1 864.1	9.0	469.0
Borcherds Quarry	664.1	18.9	251.7
Camps Bay	37.5	3.8	66.2
Cape Flats	3 082.6	50.0	982.8
Fisantekraal	269.3	2.1	200.2
Gordons Bay	154.9	6.1	42.7
Green Point	370.9	14.1	234.8
Hout Bay	215.1	4.7	102.3
Klipheuwel	1.1	0.4	3.3
Kraaifontein	371.4	7.5	133.5
Llandudno	15.0	0.5	6.8
Macassar	1 369.5	17.5	382.1
Melkbosstrand	151.2	15.4	65.3
Mitchells Plain	928.8	12.0	319.3
Philadelphia	4.2		3.3
Potsdam	1 056.9	34.6	330.6
Scottsdene	194.9	4.1	118.7
Simonstown	116.0	11.8	52.0
Wesfleur	97.9	2.3	70.6
Wesfleur Residential	272.9	2.7	87.8
Wildevoelvlei	347.6	6.8	133.5
Zandvliet	2 253.0	11.1	456.2
TOTAL	30 396.4	262.2	5 117.6

3.5 Population Growth

Cape Town is one of South Africa's fastest growing cities. It reportedly has an average growth rate of 2.57% (du Toit, 2019). A major component of this growth is caused by the expansion of informal settlements in the Cape Flats. A huge influx of Xhosa speaking people has occurred in the past number of decades. In part driven by economic difficulties in the Eastern Cape and also due to the long term drought, which has recently caused the Eastern Cape to be declared a disaster region by the government (Ellis, 2019).

Displayed in Figure 3.13 is an areal depiction of the growth of Khayelitsha through the period spanning 1984 to the present day. This figure was captured from Google Earth Pro. The areas delineated by the pink, yellow and green boundaries respectively represent the areal extent of Khayelitsha in the years of 1984 (founding year), 2011 (most recent census) and 2019 (present day). Displayed in Table 3.2 is the areal and population growth of Khayelitsha. Initially designated for 30 000 people, Khayelitsha's population has exploded.

Table 3.2 Khayelitsha Population Growth 1984 - 2019 (Strategic Development Information and GIS Department, 2013)

Khayelitsha Population Density and Growth				
Year	**Population**	**Surface Area (*km^2*)**	**Areal Density (*m^2/person*)**	**Growth Rate (*%*)**
1984	30000	4.78	159	
2011	391749	23	59	10
2019 (Low)	479922	25.4	53	2.6
2019 (Medium)	578746	25.4	44	5
2019 (High)	843073	25.4	30	10

The 2011 growth rate is based on a calculation of compound growth over the 27 years from 1984 until 2011. Due to a lack of more recent census data, further calculations were performed with three growth rates of 10, 5 and 2.57% compounded annually. The growth rates respectively correspond to the historically calculated growth rate (high boundary), half

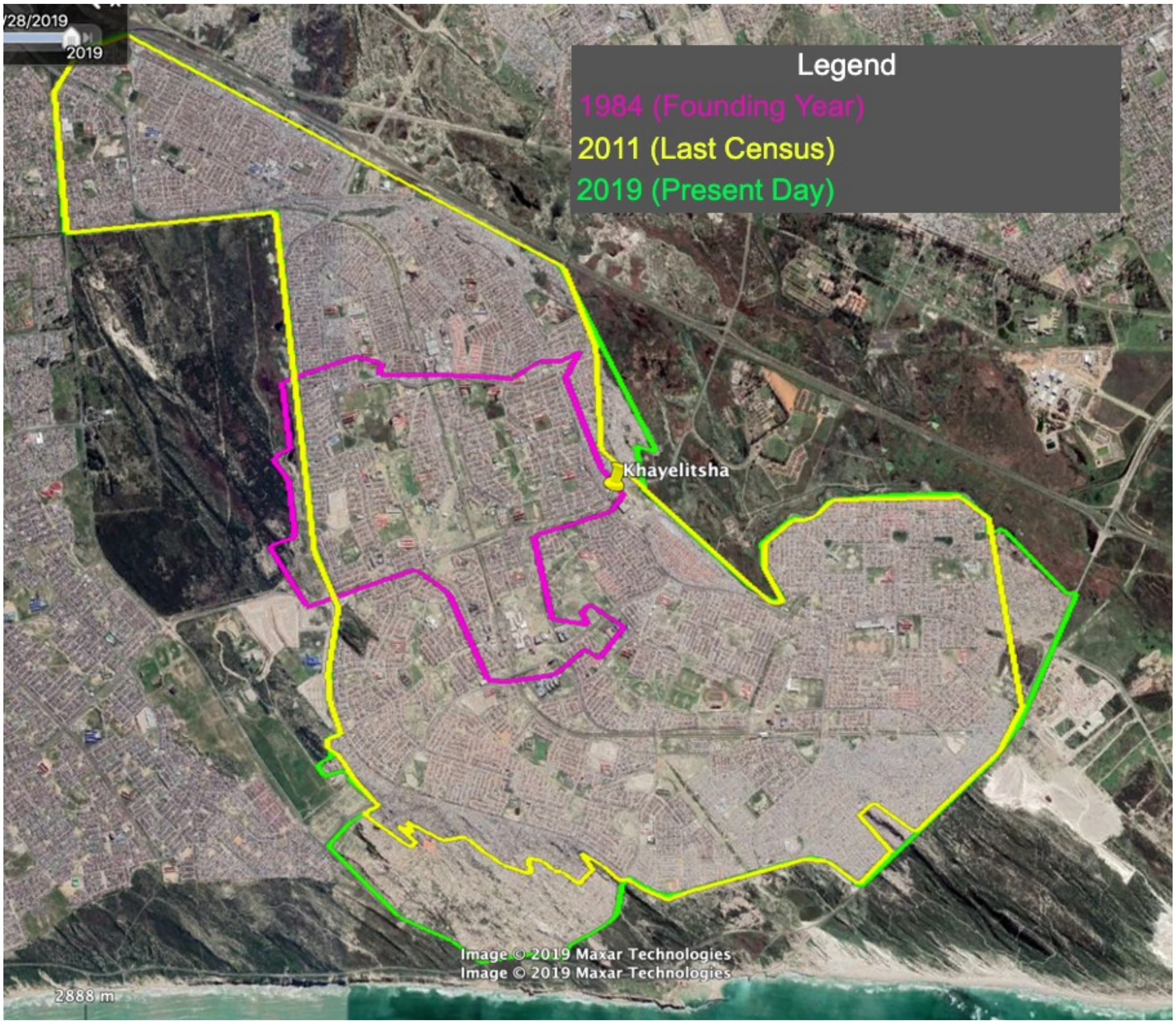

Figure 3.13 Khayelitsha Areal Growth 1984 - 2019 (Google Earth Pro, 2019

the historical growth rate (medium boundary) and the average growth rate for the entire city of Cape Town.

With recollection of the fact that due to urban migration Cape Town has one of the highest growth rates in the country. Khayelitsha has historically been growing at approximately four times the rate of Cape Town as a whole. The implications of this on sanitation systems such as drainage networks and wastewater treatment plants is that during the design phase of these systems an estimated future growth rate would have been calculated and applied to compensate for future population increase. However, if the real population growth rate was larger than the design growth rate then the systems will have been underdesigned.

This means that for wastewater treatment plants there will be more wastewater users generating wastewater than had been planned for. The result will be excessive flows being generated by the drainage region of users for a given wastewater treatment plant. Either capacity violations will occur or else the excess wastewater will have to be routed into another wastewater treatment plant.

In terms of drainage network functionality, population saturation in informal settlements leads to increased infrastructural vandalism through construction of sullage trenches, manhole lid theft and litter dumping into the conduit system. The combination of litter dumping and sediment intrusion through damaged system seals results in an increasing number of blockages.

3.6 Blockage Patterns

In a later section, the blockage patterns of the city of Cape Town will be analysed. Part of this analysis is dedicated to exploring physical causes of blockage data. However, for the purposes of this section it is necessary to acknowledge the relationship between population growth and steadily increasing blockage numbers. While it cannot be said that this growth in blockage numbers is solely attributable to population growth, it undoubtedly does have a significant effect. To this end Figure 3.14 was assembled to visually illustrate the areal correlation between human centers of activity and blockage patterns.

What these figures serve to show is that as the population grows and spreads geographically it is followed closely by blockages in the sewer network. The more saturated the population of an area is the more saturated the number of blockages in that area will be too.

In the world of art there exists a format of display called a triptych. Essentially the purpose of a triptych is to tell a visual short story in a series of three works. Triptychs are both aesthetically pleasing and powerful at showing an in-depth summary of visual information.

Figure 3.14 is a triptych of blockage data points recorded over a six-year period from 2012 to 2018, plotted geographically against a topographic StreetMap backdrop. Each of the frist two panels has a green rectangle in it showing the position of the next panel.

Each blockage event was plotted as a red cross. Over the course of years, hundreds of thousands of points were generated. This leads to a visual pattern forming of peculiarly shaped geometries overlaying the city. Upon closer inspection, these geometries begin to take form as close shadow systems of marked infrastructures (such as roads). Seemingly wherever there is human activity there have been recordings of blockages.

In Chapter 4, the information presented in this triptych will be unpacked and analysed further. For now, it serves merely as a tool for conceptualisation of the extent of the problem, which is blockage patterns in the drainage network of the city of Cape Town's sanitation system.

Note the inclusion of the furthest north area of recorded blockages. This is the area of Atlantis. For reasons of image scaling, this area could have been excluded from representation. However, in a later section it will become apparent that this areal outlier cannot be excluded, due to its blockage patterns being a significant contributor to the total blockage numbers provided in the dataset.

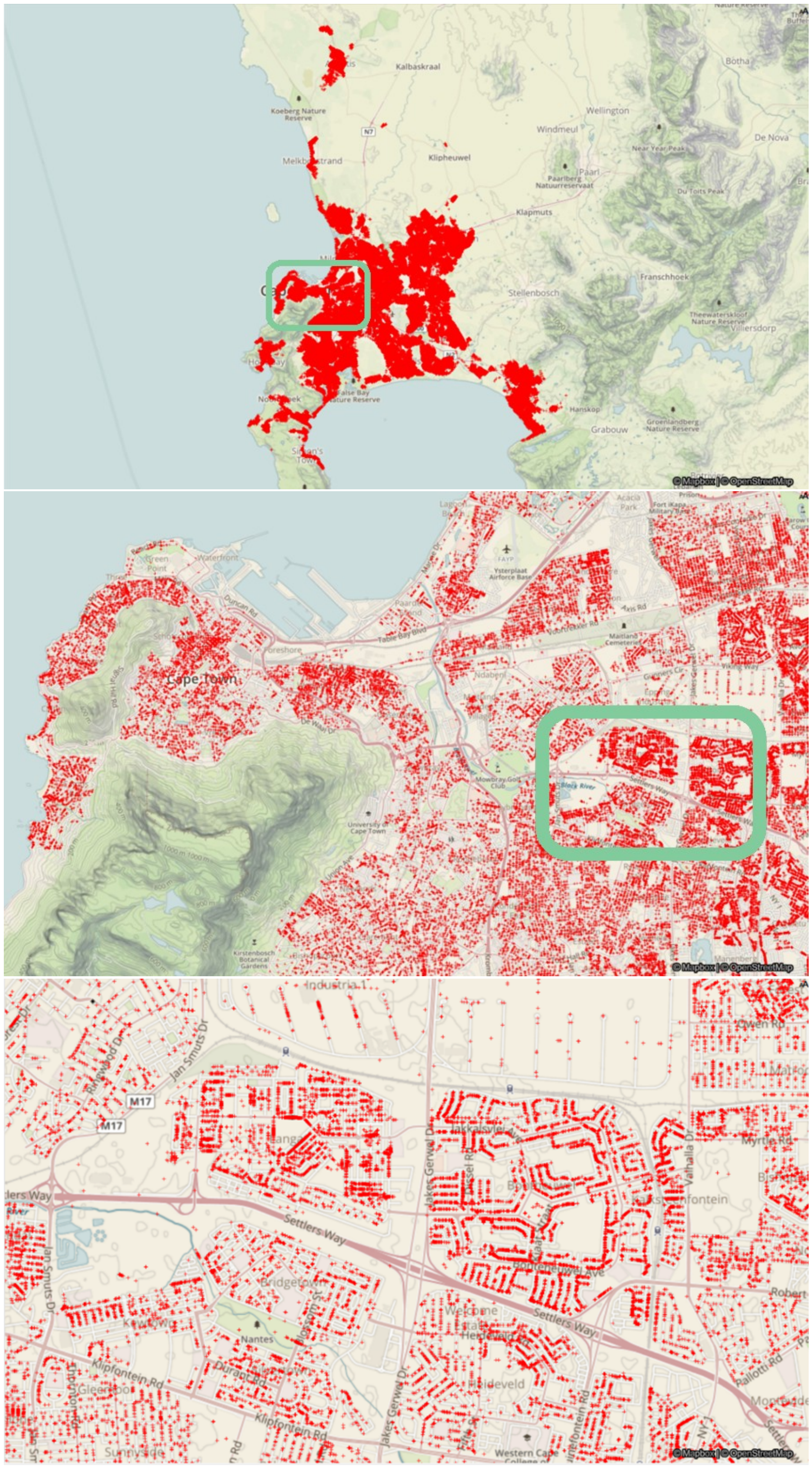

Figure 3.14 Blockage Pattern Triptych (GLS Consulting, 2019)

Chapter Four

Macro-patterns

4.1 Wastewater Treatment System

4.1.1 Pollutant Counts

Wastewater treatment plants are an essential component of any society's continued wellbeing. To continue the analogy of a society as a living organism with its various infrastructures as organs, the wastewater treatment plants are roughly equivalent to the kidney or the liver in function. They address some of the most toxic of substances generated in the 'pursuit of happiness' by society. If the kidneys and liver fail to detoxify the body, then firstly the blood and later other organs sicken and die.

The activities of a given society produce toxic waste, which is in large part collected by the drainage network and thereafter either flows under effects of gravity or is pumped into the wastewater treatment plants. This influx of waterborne toxic waste is termed influent.

The general steps of the wastewater treatment plant process are presented schematically in Figure 4.1.

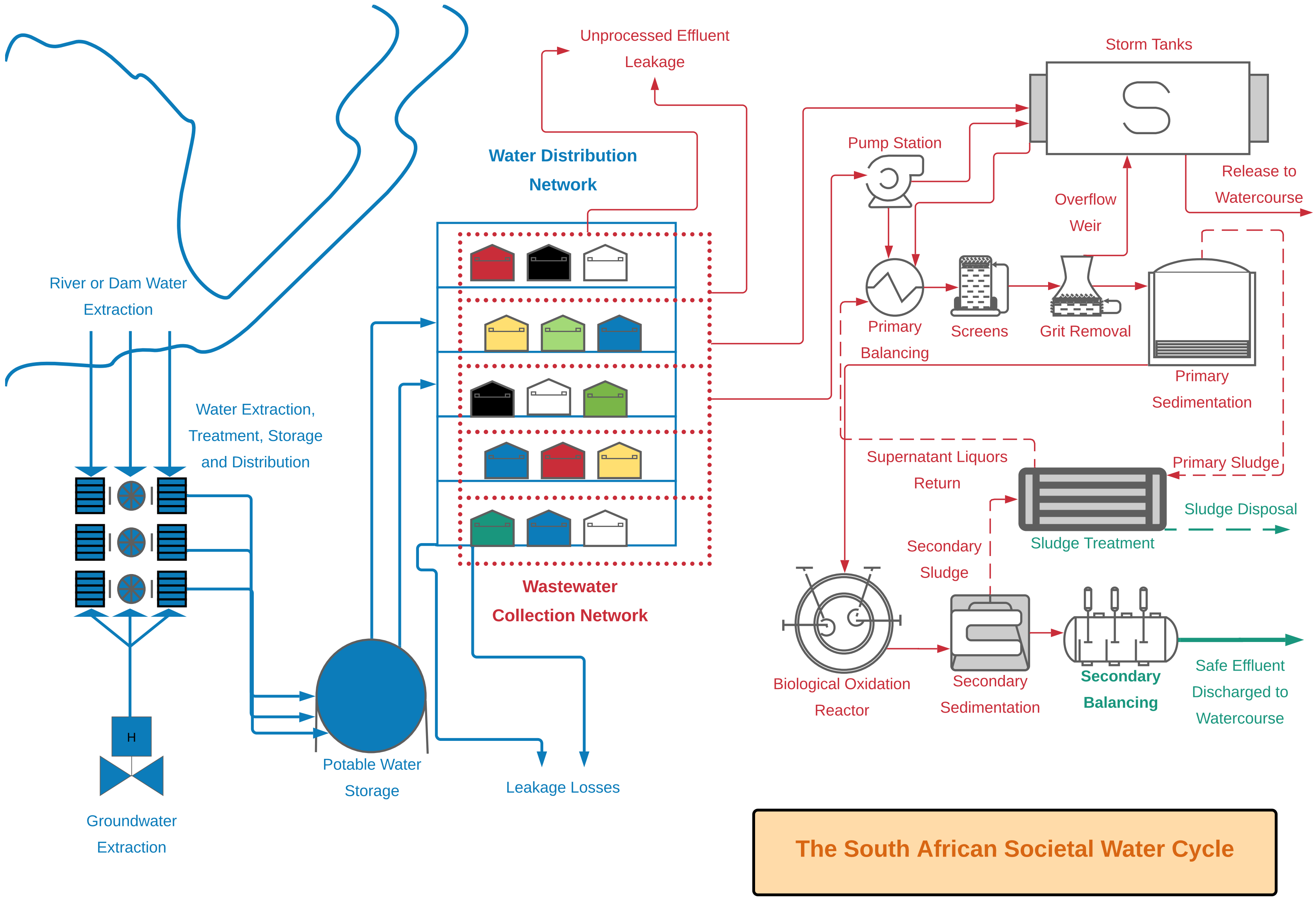

Figure 4.1 South African Societal Water Cycle

Wastewater treatment plants are carefully balanced systems wherein control loops are used to regulate processes of settling, sedimentation and organic digestion. These processes are primarily controlled by the flowrate of wastewater passing through any particular step of the wastewater treatment plant. The flowrate is directly linked to the retention time within a stage and the consequent quality of discharge.

While wastewater treatment plants are designed to handle loadings during high flow events, they are still susceptible to overloading. In a fully functional wastewater treatment plant the point of entry into the system is the storm tanks. These allow for flow attenuation so that a suitable flowrate may be maintained.

It is concerning to note that a visual inspection of Zandvliet wastewater treatment plant shows no storm tank capacity. This means that the processes of the plant are susceptible to overloading during high flow events. The Zandvliet wastewater treatment plant has attracted significant attention historically for inadequate performance (Zutari, 2020). Consequently it is undergoing an rehabilitation with the aim of increasing the capacity from 72 ML/d to 90 ML/d.

Following treatment in the wastewater treatment plant the effluent of detoxified waterborne waste is then discharged back into the aquatic surrounds of the society. Three major forms of effluent disposal are prevalent; discharge into a river system, discharge into a marine system or discharge into a groundwater system.

All three of these effluent discharge techniques are subject to a fundamental constraint that the discharged volumes must be in a 'safe' format, to prevent damage to the environment into which they are discharged. The underlying reason for this fundamental constraint is that the environments into which the effluent is discharged are without exception vital components of a significant life support system. This significant life support system is known as the hydrosphere (encompassing all forms of water existent in the confines of this planet).

If wastewater is either discharged in an inadequately treated or completely untreated form, then it serves to poison the region of the hydrosphere into which it is discharged. This is unhealthy for the life forms, which are reliant upon 'clean' water found in that particular environment.

South Africa has a finite number of wastewater treatment plants. Each of these has the invaluable function of converting the toxic influent produced by society and then to discharge a safe effluent into the environment. When this detoxification task is performed in a substandard manner, it places all downstream life forms at risk.

The city of Cape Town is the case study, which has been chosen for this research paper. The city of Cape Town has a finite number of wastewater treatment plants too. These wastewater treatment plants cater to a finite population who live within the geographical confines of the city. As the population grows, the resultant growth in volume of waterborne waste generated affects the functioning of the wastewater treatment plants. In recent years, there has been an increasing number of complaints from the population of the city of Cape Town, that adverse health effects are occurring and the alleged cause is water contamination from wastewater treatment plants (Green *et al.*, 2018).

These complaints have led to a number of researchers undertaking projects to define the ambient level of contaminants, both biological and abiological, found in the surrounding hydrospheres of the city. The results of some of these projects have been published both scientifically (Olujimi *et al.*, 2015 and CSIR, 2017b) and through news media (Green *et al.*, 2018, 2019b, 2019a).

The scientific publications respectively pertain to variability of levels of heavy metals found in the river system of Cape Town, and to the monitoring of sea outfalls and its effects on the marine environment. The news publications were created by and refer to the scientific works released by an independent research team. This research team comprised of members from the University of Cape Town, the University of Stellenbosch, the University of the Western Cape and the National University of Buenos Aires. This research team undertook to measure and report on 'indicator organisms' such as *E. Coli* and *Enterococcus*, as well as the presence of chemical contaminants such as *perfluorodecanoic acid* and *ammonia*.

Officials of the City of Cape Town, using selected justifications taken from the aforementioned CSIR report, have vociferously opposed the news media publications. These officials maintained that recorded samples of wastewater have levels of contaminants, which fall within the legal ambit of the South African National Guidelines for Water Quality (SANGWQ). Any samples, which exceed the SANGWQ limitations, have been downplayed as 'point samples not necessarily applicable to judgment on the efficacy of the wastewater treatment plants' or misrepresentative due to not acknowledging other sources of contaminants, such as but not limited to runoff or watercourse dumping.

However, presented in Tables 4.1, 4.2 and 4.3 are the respective pollutant counts from the research of Olujimi *et al.* (2015) and Green *et al.* (2018, 2019a, 2019b).

The values displayed in the first two tables are based on the range of seasonal average values measured by Olujimi *et al.* (2015). In order to cater to a high precision of data collection, these tests were conducted seasonally (4 times a year). Each site was sampled, then each sample was tested in triplicate and the average of the triplicate was recorded as the value for a given site in a given season. This methodology prevents any extreme outlier manipulation of the data. It is unequivocally concerning that the chemical pollutant counts for *cadmium*, *chromium*, *lead* and *mercury* are significantly higher than the guideline values of both the WHO and the SANGWQ.

The data presented in Table 4.3 is specific to the proximal hydrosphere of the Zandvliet wastewater treatment plant. The measurements were taken on the 27th of November 2018. The sampling sites were the wastewater treatment plant discharge point, exit of the Khayelit-

Table 4.1 Cape Town Hydrosphere Chemical Tests versus WHO Guidelines (Olujimi *et al.*, 2015; World Health Organisation, 2019)

Chemical	Cape Town ($\mu g/L$)		WHO Guideline	Pecentage WHO Guideline	
	Low	High		Low	High
Arsenic	1.6	13.7	10	16	137
Cadmium	1.4	8.0	3	46.7	266.7
Chromium	16.2	206.6	50	32.4	413.2
Cobalt	1.0	3.6	None Given		
Copper	18.2	120.5	2000	0.9	6.0
Lead	17.6	52.9	10	176.0	529.0
Mercury	1.5	2.6	6	25.0	43.3
Nickel	27.6	106.4	70	39.4	152.0
Zinc	172.8	722.1	500	34.6	144.4

sha wetland, the end of the the discharge furrow of the Zandvliet wastewater treatment plant, a mud sample from the Kuilsrivier and the Sandvlei bridge, the confluence of the Eersterivier and Kuilsrivier, the Sandvlei mosque and a groundwater sample (control test).

The recommended limit for *E. Coli* is zero colony forming units (*cfu*) should be detected in

Table 4.2 Cape Town Hydrosphere Chemical Tests versus SANGWQ (Hodgson and Manus, 2006; Olujimi *et al.*, 2015)

Chemical	Cape Town ($\mu g/L$)		DWQF Guideline	Pecentage DWQF Guideline	
	Low	High		Low	High
Arsenic	1.6	13.7	10	16	137
Cadmium	1.4	8.0	5	28.0	160.0
Chromium	16.2	206.6	100	16.2	206.6
Cobalt	1.0	3.6	500	0.2	0.7
Copper	18.2	120.5	1000	1.8	12.1
Lead	17.6	52.9	20	88.0	264.5
Mercury	1.5	2.6	1	150.0	260.0
Nickel	27.6	106.4	150	18.4	70.9
Zinc	172.8	722.1	500	34.6	144.4

Table 4.3 Cape Town Hydrosphere Biological Pollutant Data (Green *et al.*, 2018, 2019a, 2019b)

Site	Pollutant Counts - November 2018		
	E. Coli *(cfu/100mL)*	Enterococcus *(cfu/100mL)*	Ammonia *(mg/L)*
Effluent Discharge Point	245		54.1
Khayelitsha Wetland	51		
End KR	600		
Kuilsrivier (mud)	200000	1275000	50.8
Sandvlei Bridge	43500	465000	39.7
Confluence	872000	1285000	37.5
Mosque	46500		
Groundwater	1		

95% of samples. The recommended limit for *ammonia* is less than 1 mg/L (Hodgson and Manus, 2006).

Furthermore, it should be noted that the waters tested by Green *et al.* (2018) are not necessarily directly ingested by humans. However, the secondary impact on livelihood through human sickness caused by contact or poisoning of livestock and agriculture can cause serious ramifications to the local community.

4.1.2 Wastewater Treatment Plant Flowrates

The following information both tabular and graphical was supplied by the City of Cape Town and adapted for the purposes of this report. The information is presented over a 20-year period from 1997 to 2017. The information is first presented as a complete table of the flowrates for the respective wastewater treatment plants. Following on from this, the combined flowrates of all the wastewater treatment plants are graphically presented.

It must be noted that the within the original data there are a number of estimations which were included when meter readings were deemed untrustworthy, and there also appear to be a number of incorrect data records. However, the overall integrity of the data-set is considered to be high and as such provides a strong basis for argument.

Furthermore, it must be noted that the perspective of this data is given in terms of the Average Daily Flowrate, which is resultant from the summation of all monthly flowrates for a given year at a given site, and then divided by 365. This was considered justifiable, because the divisional difference between 365 and 366 days is negligible over the size of the data-set.

Table 4.4 is of the average daily flowrates for the wastewater treatment plants where, average daily flowrates exceeding the smaller of the Rated Hydraulic or Design Capacity are highlighted in red. Table 4.5 summarises the dates of commission, capacities and treatment process of the various wastewater treatment plants (City of Cape Town, 2018).

Two significant points of interest in Table 4.5 are; the age of the wastewater treatment plant fleet and the discrepancy between the Designed Capacity and Licensed Capacity.

With only two new wastewater treatment plants having been built in the past 20 years, one begins to wonder at the efficacy of a fleet which services a city with such a strong population growth. However, there has been an increase in capacity of the wastewater treatment plant fleet of Cape Town. This increase is planned through existing wastewater treatment plant rehabilitation and upgrades. The city has secured a loan of some R 1.3 billion to assist in its efforts to rehabilitate and upgrade the wastewater treatment plants (ESI Africa, 2020).

The wastewater treatment plants which stand to benefit from this loan are Zandvliet, Cape Flats, Bellville, Macassar, Potsdam, Melkbosstrand, Mitchells Plain, Borchard's Quarry, Hout Bay, Scottsdene, Wildevoevlei and Gordon's Bay.

From Table 4.4 it is evident that 195 out of 520 entries (37.5%) of the average daily flowrates

Table 4.4 Wastewater Treatment Plant Average Daily Flowrates in Violation of Capacity - 100% Efficiency

Treatment Works or Outfall	1997/ 1998	1998/ 1999	1999/ 2000	2000/ 2001	2001/ 2002	2002/ 2003	2003/ 2004	2004/ 2005	2005/ 2006	2006/ 2007	2007/ 2008	2008/ 2009	2009/ 2010	2010/ 2011	2011/ 2012	2012/ 2013	2013/ 2014	2014/ 2015	2015/ 2016	2016/ 2017
Athlone	105.2	100.8	96.6	75.3	89.7	83.6	89.5	85.9	103.5	109.2	119.4	120.9	122.9	111.8	121.6	134.3	138.7	124.6	113.6	97.7
Bellville	46.4	47.1	48.7	47.6	62.2	57.6	53.1	50.2	50.1	54.6	52.3	58.4	63.0	54.8	50.8	53.7	59.2	51.2	39.8	38.1
Borcherd's Quarry	25.7	24.2	23.4	22.9	30.3	30.5	26.6	32.3	31.1	31.6	33.0	35.2	33.8	36.2	35.1	36.0	39.9	36.3	39.8	34.6
Camps Bay Outfall	2.2	2.0	1.9	1.8	2.1	2.2	2.1	2.1	2.0	2.2	2.3	2.5	2.4	2.2	2.1	2.2	2.6	2.3	1.7	2.0
Cape Flats	151.2	147.3	147.4	172.6	169.1	137.5	129.3	152.0	138.6	151.0	172.5	176.2	198.7	137.8	108.5	114.8	123.3	122.8	118.9	114.2
Fisantekraal	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.7	9.3	13.4	12.1	10.8	10.4
Gordon's Bay	1.9	1.8	2.1	2.2	2.7	2.6	2.7	2.7	2.6	2.9	3.3	3.7	3.4	3.3	3.6	4.4	4.7	3.8	3.6	3.1
Green Point Outfall	27.2	26.2	25.3	25.9	26.7	24.5	26.0	27.9	26.9	27.0	29.0	28.0	28.1	27.6	26.9	27.8	29.7	27.1	21.0	26.2
Groot Springfontein	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Hout Bay Outfall	4.5	4.3	3.8	3.9	4.6	4.0	4.0	3.4	3.9	4.7	6.3	5.9	5.6	5.3	5.2	5.0	5.2	5.9	4.1	4.9
Klipheuwel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.2	0.2
Kraaifontein	5.2	5.3	5.8	5.5	7.9	7.3	8.0	10.6	16.3	20.1	20.5	21.5	21.5	26.9	19.3	19.5	14.9	12.9	10.7	6.0
Llandudno	0.5	0.2	0.2	0.1	0.2	0.3	0.2	0.2	0.2	0.4	0.2	0.3	0.2	0.2	0.2	0.2	0.2	0.2	2.5	0.2
Macassar	26.4	30.7	38.7	33.9	38.2	39.9	37.3	41.3	39.9	36.2	31.3	33.5	34.5	30.0	30.8	37.6	39.8	36.8	32.9	27.6
Melkbosstrand	1.4	1.5	1.7	1.6	1.9	1.8	2.3	2.7	2.7	3.1	3.3	3.5	3.7	3.8	3.8	3.8	4.4	3.8	3.7	3.1
Miller's Point	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Mitchell's Plain	29.5	29.3	30.3	28.7	32.1	30.6	30.8	32.1	32.6	32.7	33.6	36.9	37.4	32.8	34.1	34.9	36.3	34.3	32.6	30.1
Oudekraal	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Parow	1.2	1.3	1.5	1.5	2.1	2.4	1.5	1.3	1.0	1.0	0.9	1.1	0.7	0.9	0.9	0.8	1.2	1.0	0.2	0.0
Potsdam	25.3	28.0	28.5	28.7	33.9	32.7	34.5	34.1	33.0	37.0	35.1	40.9	45.0	44.1	46.6	52.8	55.3	51.2	46.1	49.2
Scottsdene	3.8	5.2	6.7	6.9	7.5	7.3	8.7	9.0	9.2	9.3	9.8	9.1	9.4	9.8	10.0	10.5	11.6	11.7	11.1	10.6
Simonstown	1.5	1.5	1.6	1.6	2.0	1.8	1.7	2.1	2.0	2.7	2.3	2.2	2.2	1.8	1.9	2.2	2.3	1.9	1.7	1.4
Wesfleur - Dom	6.6	5.7	5.8	5.7	6.2	5.7	6.1	5.9	5.9	6.0	5.7	6.1	6.5	6.4	7.4	8.5	11.6	10.3	9.8	10.2
Wesfleur - Ind	4.0	4.6	4.5	4.2	4.6	4.8	5.3	4.9	3.9	3.8	3.9	3.7	3.7	3.2	2.7	2.1	0.2	0.0	0.0	0.0
Wildevoelvlei	7.1	7.2	7.1	7.2	8.4	8.5	9.2	10.6	9.8	10.3	11.6	11.4	11.3	9.4	7.9	9.9	12.5	10.7	6.7	8.4
Zandvliet	47.5	48.6	48.3	48.2	46.9	43.3	48.0	51.5	49.3	54.2	59.5	68.3	71.1	73.0	74.5	84.1	92.3	96.8	97.7	82.4
Total	524.2	523.0	530.0	526.3	579.6	529.0	526.9	562.8	564.6	599.7	635.6	669.5	705.1	621.2	597.7	654.8	699.4	658.0	609.3	560.7

for the respective years, violated the minimum between the design and licensed capacities.

Any one of these violations of the capacities is concerning, because it implies that on an average day in a given year, the wastewater treatment plant experienced a mass flux greater than its capacity. The only reasonable conclusion of this is that the wastewater treatment

Table 4.5 Wastewater Treatment Plant Summary of Date of Commission, Design Capacity, Licensed Capacity and Plant Type

Treatment Works or Outfall	Commissioned	Capacity (*ML/d*)	DWS License (*ML/d*)	Plant Type
Athlone	1923	105	110	Activated Sludge
Bellville	1950	54.6	56	Activated Sludge
Borcherds Quarry	1973	35	35.3	Activated Sludge
Camps Bay Outfall	1977	5.5	2.3	Sea Outfall
Cape Flats	1960	200	161	Activated Sludge
Fisantekraal	2012	24	58	Activated Sludge
Gordons Bay	1994	3.1	3.4	Activated Sludge
Green Point Outfall	1993	40	27.3	Sea Outfall
Groot Springfontein	1984	0.01	0.01	Oxidation Pond
Hout Bay Outfall	1993	9.8	5.2	Sea Outfall
Klipheuwel	2000	0.07	0.07	Rotating Bio Disc
Kraaifontein	1971	17.5	28	Activated Sludge
Llandudno	1973	0.28	0.2	Rotating Bio Disc
Macassar	1978	38	30.7	Activated Sludge
Melkbosstrand	1977	5.4	3.6	Activated Sludge
Miller's Point	1996	0.06	TBD	Rotating Bio Disc
Mitchell's Plain	1976	45	35.3	Activated Sludge
Oudekraal	1996	0.03	TBD	Rotating Bio Disc
Parow	1976	1.2	0.8	Decommissioned
Potsdam	1957	47	43.9	Activated Sludge
Scottsdene	1976	12.5	10	Activated Sludge
Simonstown	1970	2.5	1.8	Bio Filters
Wesfleur - Dom	1978	8	6.9	Activated Sludge
Wesfleur - Ind	1978	6	3.2	Activated Sludge
Wildevoelvlei	1976	14	5.8	Activated Sludge
Zandvliet	1989	72	73.6	Activated Sludge
Total		746.55	702.38	

plants could not conceivably have adequately detoxified the wastewater, thereby rendering some proportion of the effluent to be unsafe and placing the receiving environment at greater risk.

It should be noted that the represented values are of arithmetic mean values for given years. Since real-world situations have amplitude fluctuations around the mean, it would imply that certain days within a problem year would have experienced flux amplitudes significantly below or significantly above the stated mean.

Furthermore, since the underlying data does not have a daily record resolution, but rather a monthly record resolution, it is possible that there may well have been periods in perceived 'safe' years where the wastewater treatment plants flowrate capacities were violated and these would not have been flagged in this analysis.

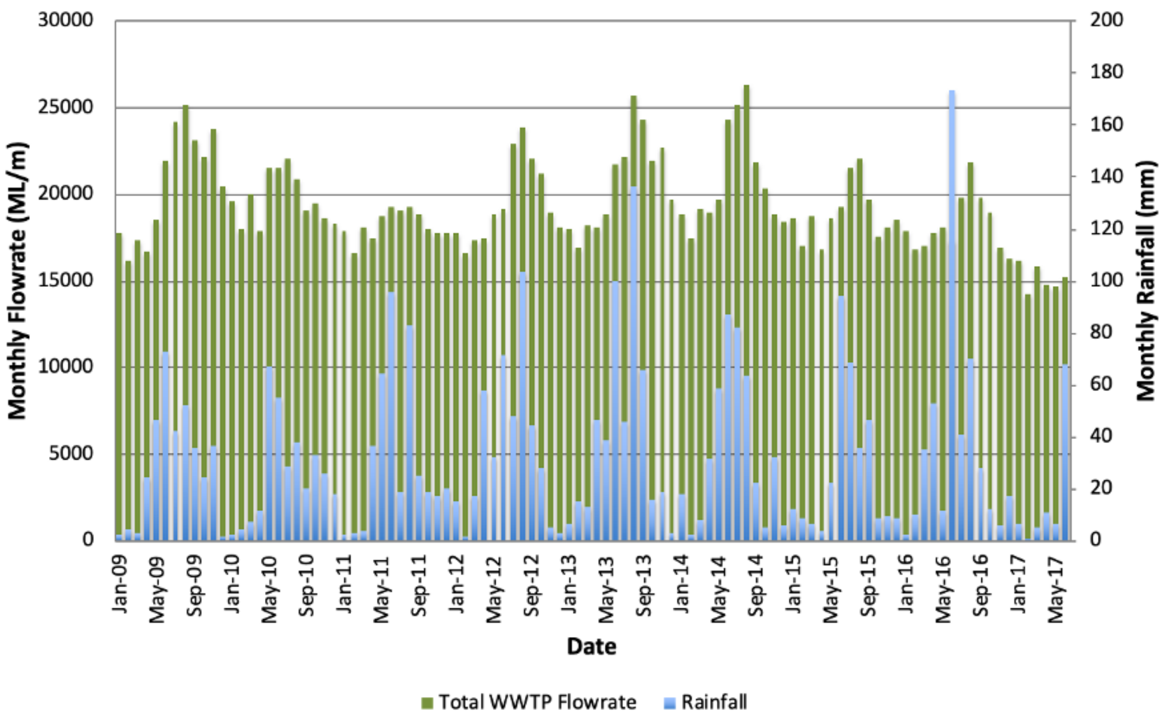

Figure 4.2 Combined Wastewater Treatment Plant Processed Flows versus Rainfall (*worldweatheronline*, 2019)

It must be noted that in Figure 4.2, the information only extends from January 2009 to May 2017, since this was the overlapping period of the data available from worldweatheronline.com and the wastewater treatment plants processed flowrates data-sets.

Shown in Figure 4.2 is a visual representation of the relationship between wastewater treatment plant flowrates and rainfall events in Cape Town. It can be seen that the peaks and troughs of the flowrates corresponds to the peaks and troughs of the rainfall events. A correlation coefficient of $r = 0.49$ was calculated showing a moderately strong correlation between the two data-sets.

This correlation between flowrates and rainfall is significant, because in periods of high rainfall the wastewater treatment plants will experience higher flowrates and initial pollutant loading spikes due to scouring of sediment deposits in the drainage network. During periods of low rainfall there will be more significant formation of sediment deposits in the drainage network. This in turn will cause increased blockage formation and consequently impede the drainage network from normal functionality in the following wet season.

From a visual analysis of Figure 4.3 it can be clearly seen that the wastewater treatment plant system of Cape Town is underperforming. This graphic is somewhat deceptive. Its bundling up of all flowrates and consequent comparison against capacities makes it seem like only 5 out of 20 years saw flowrate violations. When in fact with reference to Tables 4.4 or 4.8 it can be clearly seen that violations are significantly higher than 25%. This assertion can be further corroborated by reviewing the site specific, wastewater treatment plant flowrate graphs for the top 11 most significant wastewater treatment plants, as presented in Appendix A.

To define this problem mathematically, a set of equations is applied to quantifying the level of system failure for the respective wastewater treatment plants. The chosen equations were taken from (Mays, 2001b).

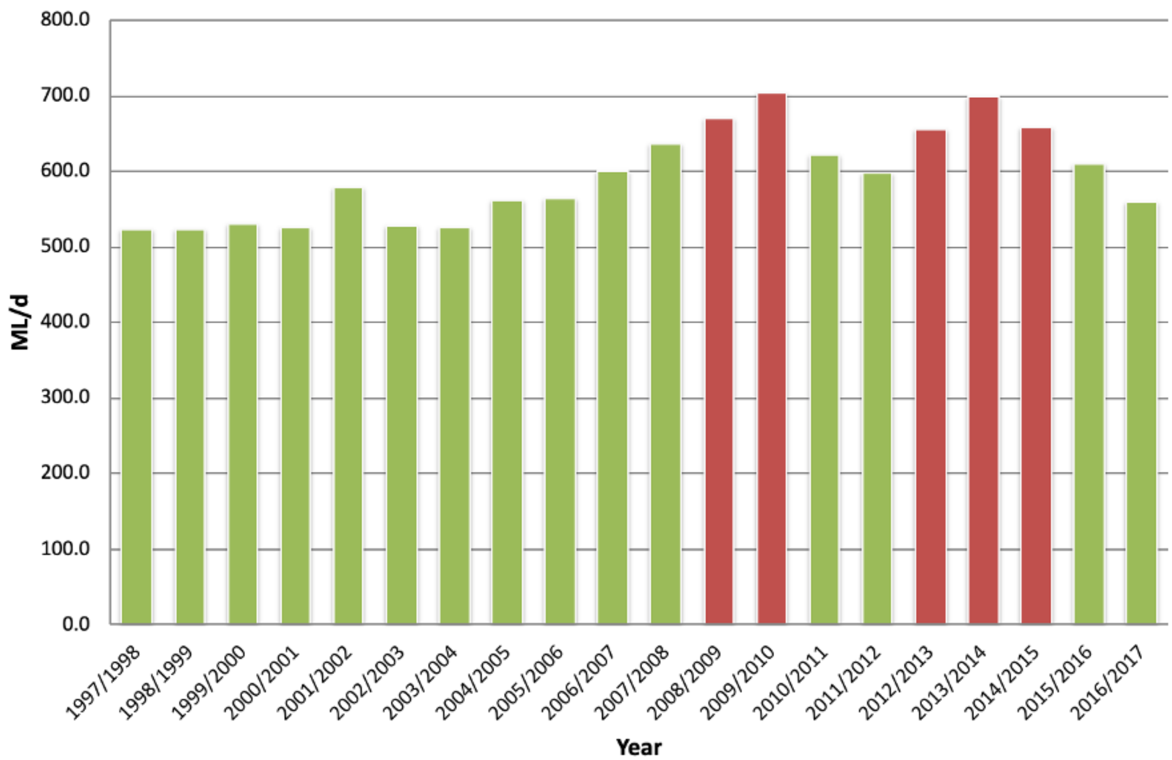

Figure 4.3 Combined Wastewater Treatment Plant Flowrates Showing Violated Years

The system reliability (p_s) is defined as the probability of non-failure, in which the system resistance exceeds the system load:

$$p_s = P_r[R > L]$$

Where:

R is the system resistance

L is the system load.

The failure probability (p_f) is defined as the inverse of the system reliability:

$$p_f = P_r[R < L] = 1 - p_s$$

The safety margin (SM), which is defined as the difference between the resistance and the load:

$$SM = R - L$$

The safety factor (SF), which is defined as, the ratio of the resistance over the load:

$$SF = \frac{R}{L}$$

For the first two equations, the system reliability was calculated as the number of years in which a given wastewater treatment plant did not fail, divided by the total number of years. For the second two equations, the resistance was taken as the minimum of the design or licensed capacity and the load, as the average flowrate experienced by each wastewater treatment plant system. The results of these calculations for each respective wastewater treatment plant are displayed in Table 4.6. It can be clearly seen from these calculations that the system reliability for the Cape Town wastewater treatment plants is alarmingly low.

A compounding factor to this insight is that these calculations were performed assuming that the wastewater treatment plants were functioning at 100% efficiency. However, it is extremely unlikely that all of these wastewater treatment plants are functioning at 100% efficiency.

One cannot simply state that the wastewater treatment plant systems are performing at a reduced efficiency, without a concept of what possible level of efficiency these could potentially be performing at. However, there was no record found of the true efficiencies of the wastewater treatment plant systems. Subsequently, a shadow system of the wastewater treatment plants was required. Due to strong similarities in function and scale, the chosen shadow system was that of the Cape Town water treatment plants.

With reference to Figure 4.1 one's attention is drawn to the relationship between water flowing into a community from the water treatment plants and the water flowing from the community into the wastewater treatment plants. Generally the flows experienced by the wastewater treatment plants are some fraction of the flows treated by the water treatment plants. The fraction is community dependent; however, for South Africa, the average return

Table 4.6 Wastewater Treatment Plant System Reliability - 100% Efficiency

Treatment Works or Outfall	Reliability (%)	Failure Probability (%)	Safety Margin	Safety Factor
Athlone	45	55	-2.24	0.98
Bellville	70	30	2.65	1.05
Borcherds Quarry	65	35	3.07	1.10
Camps Bay Outfall	85	15	0.16	1.07
Cape Flats	75	25	16.81	1.12
Fisantekraal	100	0	21.02	8.05
Gordons Bay	55	45	0.05	1.02
Green Point Outfall	65	35	0.55	1.02
Groot Springfontein	100	0	0.01	14.75
Hout Bay Outfall	65	35	0.46	1.10
Klipheuwel	75	25	0.04	2.03
Kraaifontein	65	35	4.21	1.32
Llandudno	40	60	-0.14	0.59
Macassar	20	80	-4.16	0.88
Melkbosstrand	65	35	0.71	1.25
Miller's Point	100	0	0.05	10.40
Mitchell's Plain	85	15	2.71	1.08
Oudekraal	100	0	0.03	19.66
Parow	15	85	-0.33	0.71
Potsdam	60	40	4.79	1.12
Scottsdene	70	30	1.14	1.13
Simonstown	40	60	-0.12	0.94
Wesfleur - Dom	70	30	-0.20	0.97
Wesfleur - Ind	30	70	-0.02	0.99
Wildevoelvlei	0	100	-3.46	0.63
Zandvliet	65	35	7.73	1.12

Table 4.7 Shadow System - Water Treatment Plant System Efficiency (City of Cape Town, 2019)

Water Treatment Works	Theoretical WTP Capacity (*ML/d*)	Actual Capacity (*ML/d*)	Operational Deficit (*ML/d*)	Operational Deficit (%)
Faure	500	450	50	10
Blackheath	400	400	0	0
Voëlvlei	273	180	93	34
Wemmershoek	270	200	70	26
Steenbras	150	130	20	13
Kloofnek	18	0	18	100
Somerset West Filters	15	15	0	0
Witzands	14	14	0	0
Brooklands	5	5	0	0
Albion Spring	4	4	0	0
Constantianek	3	3	0	0
Silwerstroom	3	2.7	0.3	10
Total	1655	1403.7	251.3	15

ratio is 0.65 (Stephenson and Barta, 2005). This means that wastewater treatment plants on average deal with 65% of water that the community used. The rest of the water is lost to such things as leaking pipes, garden watering and car washing on the street. However, it must not be forgotten that allowance has to be made in sewer conduit design for between 15 - 30% extra flow volume due to storm water ingress (Stephenson and Barta, 2005).

From Table 4.7 it can be seen that the average, actual efficiency for the Cape Town water treatment plant system is approximately 85% of the designed efficiency. Therefore, using a reduction of 15% on wastewater treatment plant capacities, Table 4.4 was recreated in terms of average daily flow violations, based on 85% efficiency, of the minimum between design capacity and licensed capacity. The results are shown in Table 4.8.

Recall that at 100% efficiency there were 195 out of 520 average daily flowrate violations. In comparison, with a 15% reduction in wastewater treatment plant efficiency, the corresponding number of average daily flowrate violations increase to 321 out of 520 (61.7%) of the average

Table 4.8 Wastewater Treatment Plant Average Daily Flowrates in Violation of Capacity - 85% Efficiency

Treatment Works or Outfall	1997/ 1998	1998/ 1999	1999/ 2000	2000/ 2001	2001/ 2002	2002/ 2003	2003/ 2004	2004/ 2005	2005/ 2006	2006/ 2007	2007/ 2008	2008/ 2009	2009/ 2010	2010/ 2011	2011/ 2012	2012/ 2013	2013/ 2014	2014/ 2015	2015/ 2016	2016/ 2017
Athlone	105.2	100.8	96.6	75.3	89.7	83.6	89.5	85.9	103.5	109.2	119.4	120.9	122.9	111.8	121.6	134.3	138.7	124.6	113.6	97.7
Bellville	46.4	47.1	48.7	47.6	62.2	57.6	53.1	50.2	50.1	54.6	52.3	58.4	63.0	54.8	50.8	53.7	59.2	51.2	39.8	38.1
Borcherd's Quarry	25.7	24.2	23.4	22.9	30.3	30.5	26.6	32.3	31.1	31.6	33.0	35.2	33.8	36.2	35.1	36.0	39.9	36.3	39.8	34.6
Camps Bay Outfall	2.2	2.0	1.9	1.8	2.1	2.2	2.1	2.1	2.0	2.2	2.3	2.5	2.4	2.2	2.1	2.2	2.6	2.3	1.7	2.0
Cape Flats	151.2	147.3	147.4	172.6	169.1	137.5	129.3	152.0	138.6	151.0	172.5	176.2	198.7	137.8	108.5	114.8	123.3	122.8	118.9	114.2
Fisantekraal	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.7	9.3	13.4	12.1	10.8	10.4
Gordon's Bay	1.9	1.8	2.1	2.2	2.7	2.6	2.7	2.7	2.6	2.9	3.3	3.7	3.4	3.3	3.6	4.4	4.7	3.8	3.6	3.1
Green Point Outfall	27.2	26.2	25.3	25.9	26.7	24.5	26.0	27.9	26.9	27.0	29.0	28.0	28.1	27.6	26.9	27.8	29.7	27.1	21.0	26.2
Groot Springfontein	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Hout Bay Outfall	4.5	4.3	3.8	3.9	4.6	4.0	4.0	3.4	3.9	4.7	6.3	5.9	5.6	5.3	5.2	5.0	5.2	5.9	4.1	4.9
Klipheuwel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.2	0.2
Kraaifontein	5.2	5.3	5.8	5.5	7.9	7.3	8.0	10.6	16.3	20.1	20.5	21.5	21.5	26.9	19.3	19.5	14.9	12.9	10.7	6.0
Llandudno	0.5	0.2	0.2	0.1	0.2	0.3	0.2	0.2	0.2	0.4	0.2	0.3	0.2	0.2	0.2	0.2	0.2	0.2	2.5	0.2
Macassar	26.4	30.7	38.7	33.9	38.2	39.9	37.3	41.3	39.9	36.2	31.3	33.5	34.5	30.0	30.8	37.6	39.8	36.8	32.9	27.6
Melkbosstrand	1.4	1.5	1.7	1.6	1.9	1.8	2.3	2.7	2.7	3.1	3.3	3.5	3.7	3.8	3.8	3.8	4.4	3.8	3.7	3.1
Miller's Point	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Mitchell's Plain	29.5	29.3	30.3	28.7	32.1	30.6	30.8	32.1	32.6	32.7	33.6	36.9	37.4	32.8	34.1	34.9	36.3	34.3	32.6	30.1
Oudekraal	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Parow	1.2	1.3	1.5	1.5	2.1	2.4	1.5	1.3	1.0	1.0	0.9	1.1	0.7	0.9	0.9	0.8	1.2	1.0	0.2	0.0
Potsdam	25.3	28.0	28.5	28.7	33.9	32.7	34.5	34.1	33.0	37.0	35.1	40.9	45.0	44.1	46.6	52.8	55.3	51.2	46.1	49.2
Scottsdene	3.8	5.2	6.7	6.9	7.5	7.3	8.7	9.0	9.2	9.3	9.8	9.1	9.4	9.8	10.0	10.5	11.6	11.7	11.1	10.6
Simonstown	1.5	1.5	1.6	1.6	2.0	1.8	1.7	2.1	2.0	2.7	2.3	2.2	2.2	1.8	1.9	2.2	2.3	1.9	1.7	1.4
Wesfleur - Dom	6.6	5.7	5.8	5.7	6.2	5.7	6.1	5.9	5.9	6.0	5.7	6.1	6.5	6.4	7.4	8.5	11.6	10.3	9.8	10.2
Wesfleur - Ind	4.0	4.6	4.5	4.2	4.6	4.8	5.3	4.9	3.9	3.8	3.9	3.7	3.7	3.2	2.7	2.1	0.2	0.0	0.0	0.0
Wildevoelvlei	7.1	7.2	7.1	7.2	8.4	8.5	9.2	10.6	9.8	10.3	11.6	11.4	11.3	9.4	7.9	9.9	12.5	10.7	6.7	8.4
Zandvliet	47.5	48.6	48.3	48.2	46.9	43.3	48.0	51.5	49.3	54.2	59.5	68.3	71.1	73.0	74.5	84.1	92.3	96.8	97.7	82.4
Total	524.8	523.0	530.0	526.3	579.6	529.0	526.9	562.8	564.6	599.7	635.6	669.5	705.1	621.2	597.7	654.8	699.4	658.0	609.3	560.7

daily flowrates violated the system capacities. Table 4.9 presents system reliabilities with 85% efficiencies assumed.

If the 85% efficiency assumption is believed, these figures are truly alarming. This argument is further validated from the viewpoint that the residents of the city are more likely to become restless over the water supply being inadequately treated. No one wants to drink fouled water. Therefore, there is greater public pressure on the water treatment plant systems to perform well. The wastewater treatment plant system output on the other hand, is recycled outside of the public sphere of interest, through the proximal hydrosphere of the city. Subsequently, underperformance of the wastewater treatment plant system will be of lesser immediate concern to the public and more likely to go unnoticed until it reaches a critical point.

However, an important consideration, which is often ignored, is that the entirety of the wastewater generated by the city must be rendered safe through some system or other. The wastewater treatment plant system is the full set of solutions, which the city has implemented. A complimentary, yet barely acknowledged, system for wastewater detoxification, is the ability of the proximal hydrosphere to absorb and detoxify the wastewater. What seems to not have been quantified or considered is the finite capacity of the proximal hydrosphere to detoxify wastewater.

Say for instance that a particularly heavy season of rain occurs in the city. Following on from this, a correspondingly high flow volume passes through the sewerage network. This in turn causes high levels of sedimentary scouring to occur, flushing out large volumes of pollutants, both chemical and biological. The wastewater treatment plants, which have been shown to generally underperform, are then laden with a higher flow and pollutant volume. Consequently, the absolute maximum possible volume of wastewater, which can be treated by the wastewater treatment plant system, is treated. The remainder of the water

Table 4.9 Wastewater Treatment Plant System Reliability - 85% Efficiency

Treatment Works or Outfall	Reliability (%)	Failure Probability (%)	Safety Margin	Safety Factor
Athlone	15	85	-17.99	0.83
Bellville	10	90	-5.54	0.89
Borcherds Quarry	25	75	-2.18	0.93
Camps Bay Outfall	15	85	-0.19	0.91
Cape Flats	35	65	-7.34	0.95
Fisantekraal	100	0	17.42	6.84
Gordons Bay	30	70	-0.42	0.86
Green Point Outfall	5	95	-3.54	0.87
Groot Springfontein	100	0	0.01	12.54
Hout Bay Outfall	40	60	-0.32	0.93
Klipheuwel	75	25	0.03	1.73
Kraaifontein	55	45	1.58	1.12
Llandudno	10	90	-0.17	0.50
Macassar	0	100	-8.77	0.75
Melkbosstrand	45	55	0.17	1.06
Miller's Point	100	0	0.05	8.84
Mitchell's Plain	15	85	-2.59	0.92
Oudekraal	100	0	0.02	16.71
Parow	10	90	-0.45	0.60
Potsdam	55	45	-1.79	0.95
Scottsdene	30	70	-0.36	0.96
Simonstown	15	85	-0.39	0.80
Wesfleur - Dom	25	75	-1.23	0.83
Wesfleur - Ind	30	70	-0.50	0.85
Wildevoelvlei	0	100	-4.33	0.53
Zandvliet	55	45	-3.07	0.95

which is in excess of the plant capacity has an unknown fate. Reportedly it is adequately treated; however, contrasting this is evidence of contamination of downstream waters from the wastewater treatment plants (Green *et al.* (2018, 2019a, 2019b).

From the point of introduction into this riverine system all the way out past its exit into the ocean, there will be chemical bleaching of the system. The result of this is that, where once there was a system of fauna and flora, which could handle the detoxification of some unkown quantity of dumped wastewater, there is now an environment almost entirely devoid of life. In effect the riverine system will become nothing more than a channel through which toxic water passes. Either to another system, which can detoxify it, or onto some other point where it will settle and remain unchanged.

Now consider if this happens on a regular cycle. The systems resilience may handle a few of these high loads but each time it becomes more and more crippled, and its innate detoxification abilities, dwindle and die.

In summary, what these flowrates mean is that there is serious credence to the arguments put forward by (Green *et al.*, 2018) that there is indeed strong contamination stemming from the WWTP systems inability to process the wastewater, generated by the city.

4.1.3 Reliance on Pumping

The topography of the City of Cape Town has both mountainous and flatland features. The majority of the population of the city lives on the flatland areas. With respect to the wastewater treatment plants this has the effect of a high proportion of flow being routed to the wastewater treatment plants through pump stations.

Pump stations rely on pumps, which in turn rely on power being delivered from a power source. The primary power supply for the Cape Town pump stations is the national power grid. Eskom almost exclusively feeds the national power grid. This is problematic considering the previously explained plight of Eskom.

It must be noted that in most engineering solutions redundancies are built into the design for when the operating conditions are not the same as the design conditions. In the case of pump stations, these redundancies will take the form of alternate power sources be they power supplied by Independent Power Producers or on site power-generating units (Van Zyl and Van Dijk, 2011).

In the case of on site power generating units, one of the primary concerns is cost of operation. The general trend in South Africa for power generating units is that they are diesel fueled. When looking at the costs incurred by Eskom's Open Cycle Turbine generators, it is clearly discernible that producing power through diesel driven power generating units is highly cost inefficient. This is concerning since using power generating units to power pump stations will dramatically increase the cost of operation of the network, which is arguably already financially constrained.

The graphics displayed in Figures 4.4, 4.5 and 4.6 were produced from data extracted from a series of PDF documents provided by GLS Consulting, displaying the schematic layout of the various catchments of the wastewater treatment plants. The data therein is derived from the

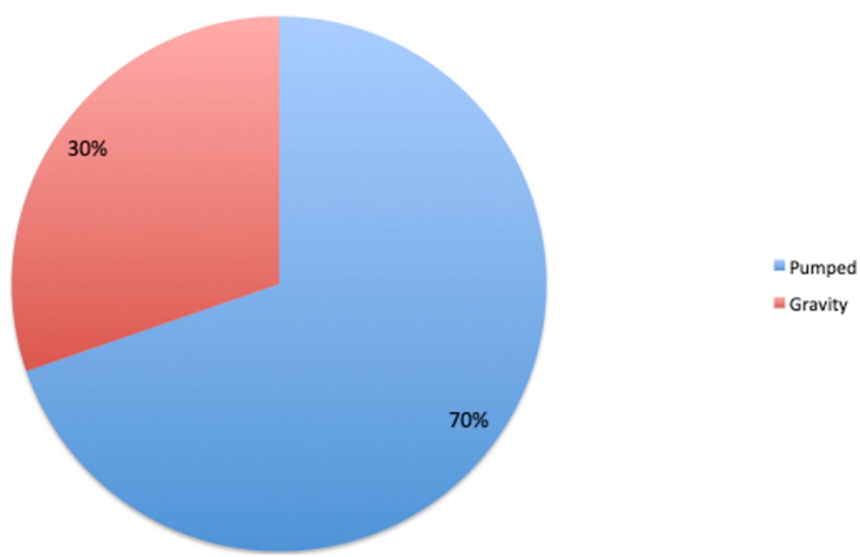

Figure 4.4 Wastewater Inflow - Pumped versus Gravity

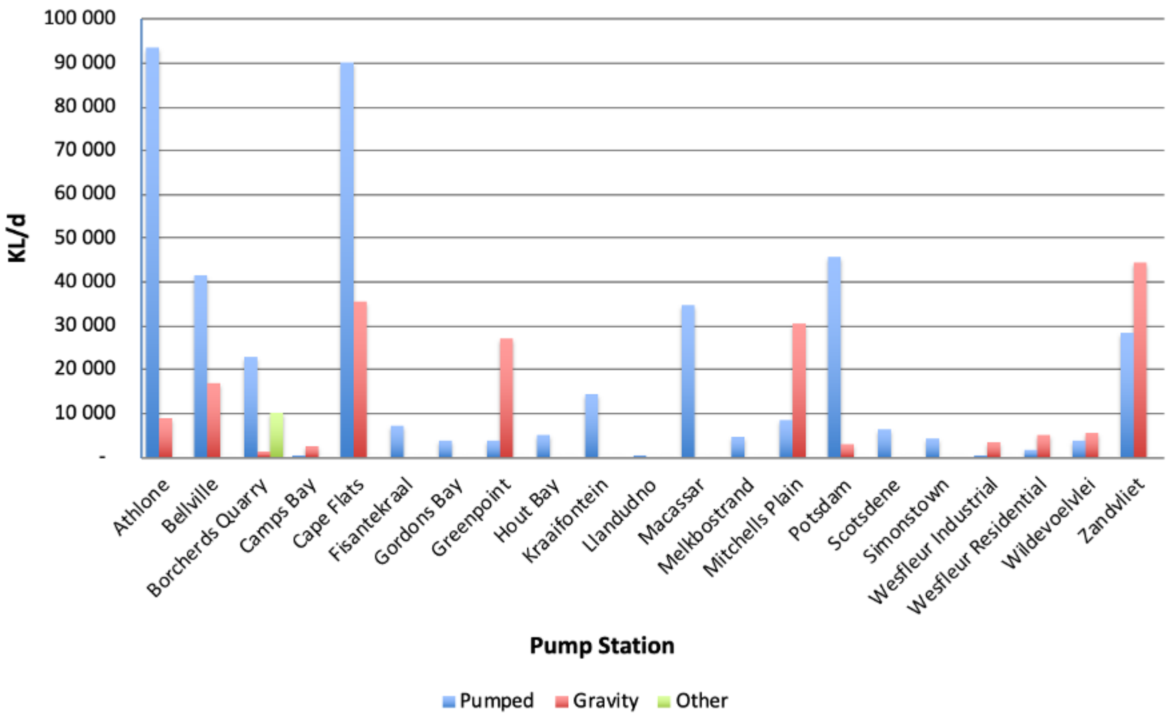

Figure 4.5 2014 Wastewater Inflow Relative to Wastewater Treatment Plant

year of 2014 and is therefore not wholly representative of trends in network management; however, it does provide a clear understanding of the city's heavy reliance on pumping stations and on a reliable power supply. The PDFs were supplied by GLS Consulting and with permission from the City of Cape Town.

Figure 4.4 presents the proportions of wastewater inflow for the entire City of Cape Town. With 70% of the total flow of the wastewater treatment plants being routed through pump stations and only 30% of flow being routed under the effects of gravity, it is clearly inferable that a serious problem would arise in the city's ability to treat its produced wastewater, should there be any sustained power outages in the national power grid.

Figures 4.5 and Figure 4.6 respectively present the site-specific inflow methods and number of pumps per site for the City of Cape Town. From these graphics it can be seen that in areas such as Athlone and the Cape Flats, where the topography tends to be flat, there is

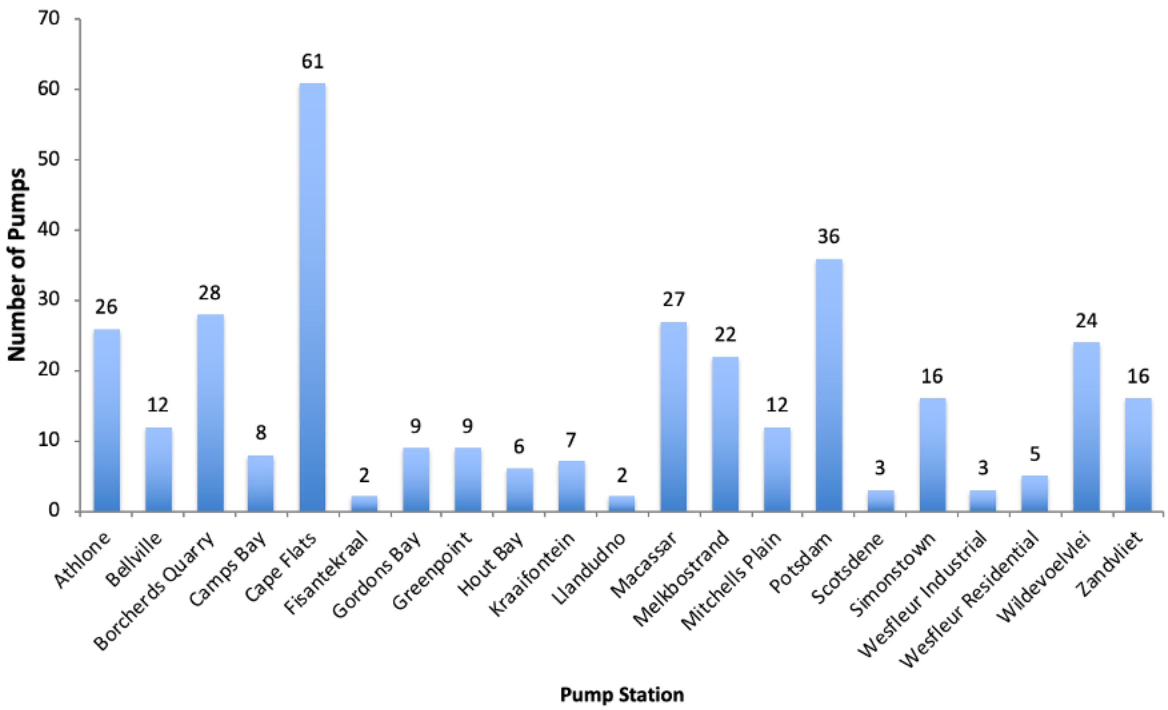

Figure 4.6 2014 Number of Pumps Relative to Wastewater Treatment Plant

an accordingly high number of pumps required to transfer the wastewater from the endpoint of the gravity operated drainage network to the wastewater treatment plants.

Furthermore, it must be noted that the flatland regions tend to be the less desirable residential sites of the city, consequently they tend to be home to the historically disenfranchised and poorer portion of the city's population. This is significant, since it is the residents of these areas, which are most at risk of being unable to help themselves when financial solutions to a given problem are required.

There exists a parallelism between the size of wastewater treatment plants and reliance on pumping. Almost all of the biggest wastewater treatment plants have a high number of pump stations to supply their requisite flowrates.

Should Cape Town face a power crisis, an already burdened fleet of wastewater treatment plants (in terms of flowrates relative to capacities) will be placed under even greater pressure, since the wastewater will become unavailable or significantly more expensive to transfer from the end points of the gravity systems through the system of rising mains into the wastewater treatment plants. A request for information was made to the City of Cape Town requesting information of standby power at the biggest wastewater treatment plants; however, the request went unanswered and so the status of standby power is unknown. Therefore it is unknown what the true risk is of power outages to the wastewater treatment plant fleet.

Should the efficiency of the wastewater treatment plant fleet come under greater pressure to perform, it is likely that these will not meet the requisite standards of performance. Contamination of all subsystems of the proximal hydrosphere becomes the only plausible outcome. In a city already faced with an enormous task of balancing water demand to high population growth rate, this, in turn, could have dire consequences on the future economic and physical health of the city and its citizens.

4.2 Drainage System Blockage Patterns

4.2.1 Introduction

It has been shown that by financial and volumetric consideration, by far the largest component of a wastewater treatment system is the drainage network. This is the section of the system, which is used for collection of the wastewater from the users, and the subsequent transfer to the wastewater treatment plants. The drainage network can be loosely broken up into two conceptual entities, that which operates under the effects of gravity - gravity network and that which operates under the effects of pumping - pressurised network.

The drainage network connects the users to the wastewater treatment plants. Considering that the users of a wastewater treatment system produce waste as a function of pursuing their livelihoods, it is therefore inconceivable that should there be a breakdown of functioning in the drainage network, that the users will stop producing waste. Instead, this waste will just accumulate wherever it settles causing something akin to a sewage traffic jam. This practical impasse means that if there is a breakdown in drainage functionality, numerous problems arise. These problems are generally observed as ground level flooding, displacement of manhole covers and possibly even pipe bursts from induced pressure.

Networks that experience blockages may become subjected to pressure. These pressure events could push wastewater out of the drainage network through cracks and imperfect pipe joints, into the surrounding ground or ground water (thereby contaminating the surroundings of the drainage network). While transient pressure surges may cause increased stress on the pipes they are generally too low in magnitude to cause any significant damage. Furthermore the upstream manholes tend to overflow alleviating pressure when a blockage occurs.

The system of an entire city is vast and complex. This research focuses on one subsystem of the super-system of the city of Cape Town, namely that of the wastewater system. The focus of this section is the drainage network of the city and its blockage events, as a reflection of the patterns of the underlying vivirithm.

4.2.2 Analysis

The data for this chapter was supplied by GLS Consulting with permission from the City of Cape Town, and each datum point in the data-set represents one blockage event, which was recorded at a specific time and place. It must be noted that the data-set has implicit discrepancies, since the events were recorded by municipal workers who were responding to a report of a blockage logged by a citizen of the city. Subsequently, there is a disjoint between the reality of the city's blockages (the exact when, where and how of the blockages is not

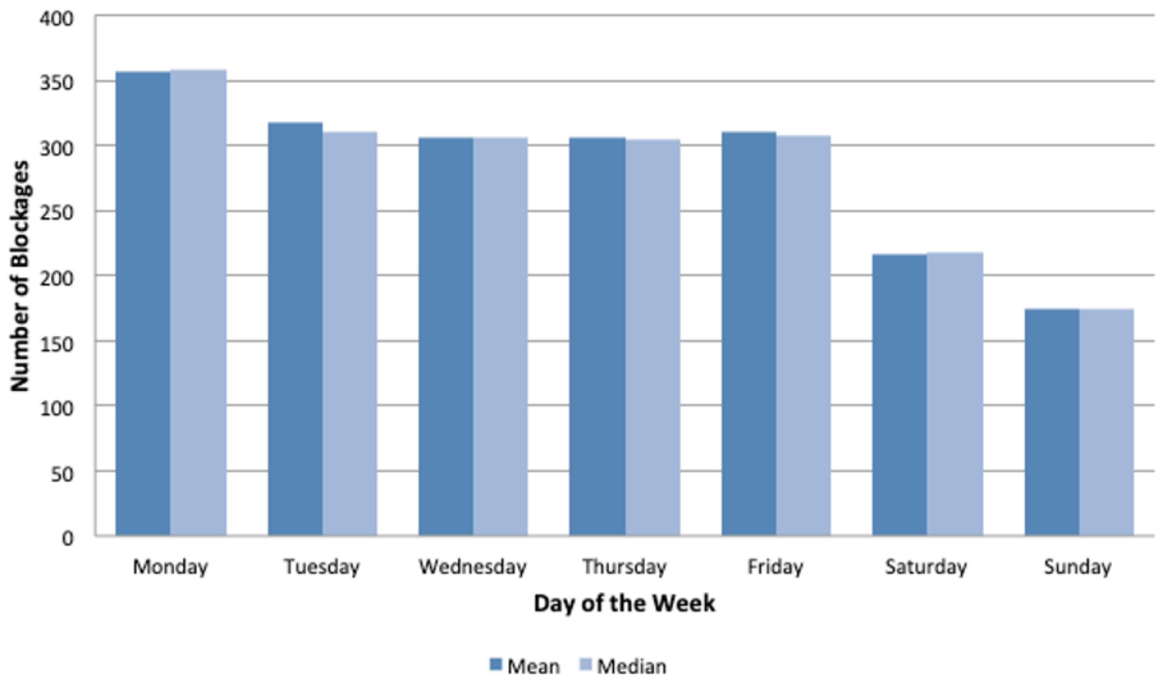

Figure 4.7 Average Blockage Events per Day of the Week

recorded but instead only the when, where and how as perceived by a municipal worker following a citizen's report). As such, this blockage data-set is a reflection of the actual blockage pattern of the vivirithm of the city, filtered through the work of municipal workers.

In order to supply some perspective of the nature of information filtering between the reality of a blockage event, and its actual record, a graph is shown in Figure 4.7 of mean and median blockage events recorded per days of the week over the analysis period. It can be clearly seen that the highest average number of blockage events was recorded on Mondays with a generally consistent number of blockage events recorded through the rest of the weekdays. A significantly lower number of blockage events were recorded on Saturdays and Sundays. This is not representative of the blockage cycle attuning itself to an anthropogenic imposed time system such as the days of the week, but rather the result of citizens less readily reporting blockages over the weekend and municipal workers being less readily available and willing to address blockage events on the weekend.

This implicit filtering being noted, it must be stated that the data-set is so large that the discrepancies between when the blockage event actually happened and when it was recorded by the city is considered negligible. Subsequently the data-set is considered to be of a high data quality.

Furthermore, efforts were made to filter the data-set to improve its integrity. The process of data filtering is explained below. The data-set spans a period of just over 10 years from 1 January 2009 until 30 April 2019 and in its unfiltered format comprises 1 080 240 data points. The data-set was compiled from 11 annual data-sets. The data-set consists of entries, each with a set of descriptors:

- Notification
- Street
- City
- Created on (denoting date of creation)
- Created at (denoting time of creation)
- Coding code txt (denoting a description of the blockage event)
- Sub Council
- Wards subcode
- X-Y Co-ordinate (denoting an X-co-ordinate)
- X-Y Co-ordinate (denoting a Y-co-ordinate)

Two forms of the total data-set were prepared. One set titled Light Filtered and one titled Heavy Filtered. In both forms the first step of data filtering was to use Microsoft Excel's 'Remove Duplicates' function to filter out any duplicate entries.

The light filtered data-set was then further filtered by sorting all of the data in an ascending order the 'street' and 'city' descriptor columns and deleting all entries, which were either missing a descriptor or had a nonsensical descriptor such as an @, £, Ò etcetera. This filtering process served to create a data-set of 1 069 988 data points. Therefore, through this light filtering process, approximately 1% of the data points were removed.

The heavy filtered data-set was further filtered by sorting data in ascending order, for each, respective descriptor column and deleting all entries, which were missing entirely or had a nonsensical descriptor in any of the descriptor columns. This filtering process served to create a data-set of 871 330 data points. Therefore, through this heavy filter process, approximately 19.3% of the data points were removed.

The purpose of the light filtered process was to minimise the impact of random or erroneous entries on data analysis. Thereby improving the analysis integrity. The purpose of the HF process was to severely prune the original data-set and subsequently to provide a counterpoint (checkpoint) set of patterns against which to evaluate conclusions drawn from the light filtered data-set. Furthermore, this HF data-set could be used as a guideline for how the blockage numbers would vary, should there be a conscious effort made to reduce blockage numbers.

The analysis presented in this chapter largely focuses on the data held within the light filtered set. In specific instances graphics from both the light filtered and heavy filtered sets will be displayed and referenced. The correlation coefficient between the light filtered and heavy filtered sets (grouped by month) was calculated at $r = 0.69$, indicating a strong positive correlation between the two sets, which was to be expected considering their shared origin.

4.2.3 Effects of Rainfall on Blockage Event Records

As explained in an earlier section, rainfall events have a strong effect on the city's patterns of water usage and wastewater generation. It is necessary for the following sections to understand the specific relationship between rainfall events, and the resulting changes in blockage event patterns.

All rainfall data was extracted from worldweatheronline.com (*worldweatheronline*, 2019), which sources its data from the Cape Town International Airport weather station. While it is acknowledged that taking rainfall patterns from one point is not wholly representative of the exact rainfall patterns of the entire area being analysed, the Cape Town International Airport is relatively central to the area under investigation and was considered to be suitable for the purposes of this report.

To this end Figure 4.8 shows the annual rainfall for the ten-year period from 2009 to 2018, which corresponds to the blockage event data-set.

Figures 4.9 and 4.10 show the relationship between rainfall and blockage events for the light filtered and heavy filtered data-sets respectively. The calculated correlation coefficients for the light filtered and heavy filtered to rainfall events were $r = 0.41$ and $r = 0.46$ respectively. These correlation coefficients show a moderate positive relationship.

An increased probability of blockage with an increase in flow volume (and movement of blockage prone matter such as foreign objects) will, in turn, lead to an increase in surcharged conditions, manhole lid displacement, pipe bursting and other readily documented drainage network blockage results. When these results are observed by the public and reported to the municipality, the blockages are recorded. This again points to the lag between actual blockage creation and blockage recording, since the blockages caused by low flow exacerbated phenomena such as sedimentation during dry periods, will precede the discovery of the

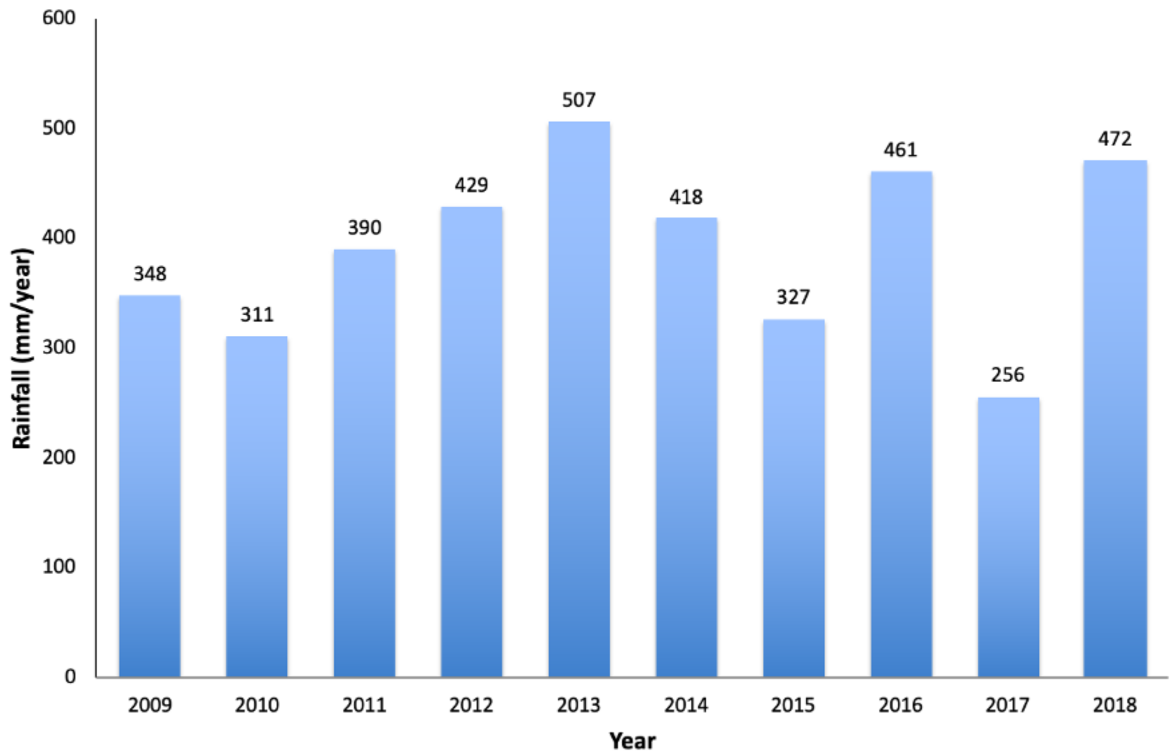

Figure 4.8 Cape Town Annual Rainfall (*worldweatheronline*, 2019)

blockage during wet periods.

A point of interest seen in the heavy filtered graphic was the severe reduction in the 2017 blockage event peaks (during the wet months). This would point to an increased level of descriptor omission in the X-Y Co-ordinate descriptors of the data-set, which in turn points to either a lower level of care taken by municipal workers when recording descriptors or to a system malfunction with the recording devices, although the former is more likely than the latter.

It must be noted further that the accuracy of the descriptor quality is based on the diligence of municipal workers, recording the blockage events in combination with, the accuracy of their devices ability to record the GPS co-ordinates of the blockage event. Effectively, the level of descriptor quality is a pseudo random function based on municipal worker effort

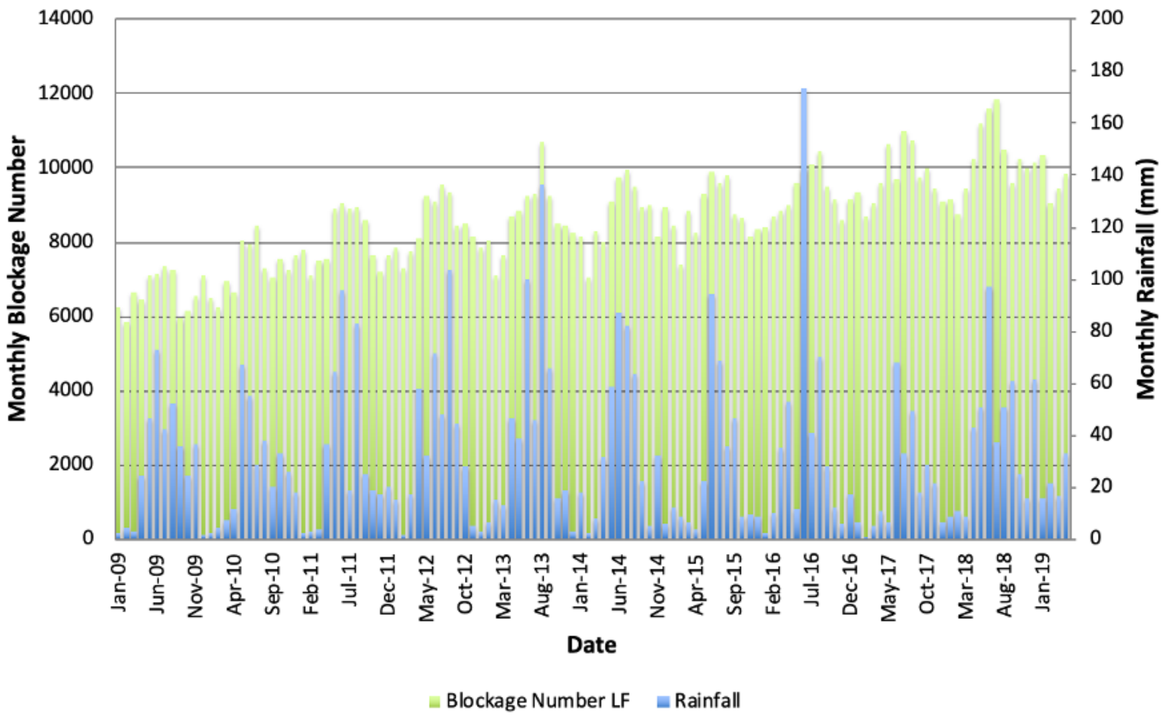

Figure 4.9 Cape Town Annual Rainfall (*worldweatheronline*, 2019)

and device accuracy. However, the data-set filtering was targeted specifically at empty or nonsensical descriptors. A review of the annual correlation coefficients for the data grouped per month was performed and an interesting pattern was noted. This pattern is presented in Table 4.10.

It can be clearly seen that for the years of 2009 through 2015 there is a very strong r-value. From 2016 through 2018 there is a decreasing strength of r-value, with the lowest recorded for the year of 2017. This warranted interest; however, was not considered to be

Table 4.10 Annual Correlation Coefficients for Light Filtered and Heavy Filtered Grouped per Month

Year	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018
Correlation	0.94	0.98	0.98	0.89	0.97	0.98	0.96	0.80	-0.26	0.43

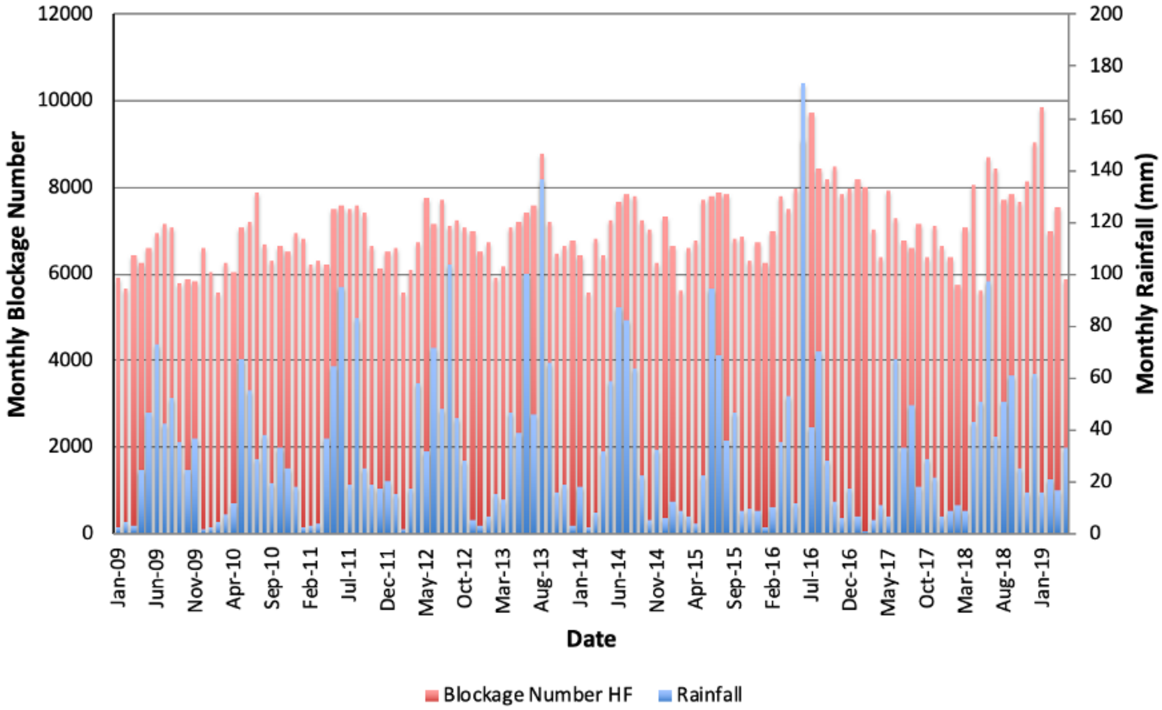

Figure 4.10 Cape Town Annual Rainfall (*worldweatheronline*, 2019)

of great importance within the scope of this report. Subsequently, it was concluded that this lowered level of descriptor accuracy, coinciding with the drought period was due to the whole municipal water system (including wastewater), being under heavy duress and the level of descriptor accuracy may have become relatively unimportant in the greater scheme of problems occurring at the time.

With respect to the lower runoff shown in Figure 4.8 there are two points of interest. During low rainfall seasons there will be a smaller volume of stormwater intrusion into the network. This will make the likelihood of conduit flooding and overflow due to blockages smaller. This will decrease the number of blockages reported in that year. Furthermore there will be a somewhat reduced amount of stromwater borne sediment intrusion into the conduit system. This will reduce the number of sediment linked blockages somewhat.

4.2.4 Blockage Causes

Sewerage blockages are caused whenever the network cross-section at a given point becomes impassable to the sewer flow. The causes of a cross section blockage are consequently numerous. The causes recorded in the given data-set are presented below:

- Cement Layers
- Collapse
- Fats
- Foreign Objects
- Rags
- Roots
- Sand
- Sewer Pump Station: Overflow
- Sewer: Blocked/Overflow
- Sewer: Pipe Broken

Provided below is a probable list of explanations for the various recorded causes of blockage.

Cement Layers can be attributed either to the dumping of cement from nearby construction sites into the sewerage network or they could stem from misplaced cement from the actual construction phase of the sewerage network section.

Collapse can be attributed to poor workmanship during construction, material aging, material corrosion through exposure to substances such as sulphur dioxide (SO_2) or from design load exceedance (such as heavily laden trucks traversing the road above a sewerage section).

Fats are a globally recognised, serious cause of sewerage network functionality breakdown. It serves to either coat the interior of the sewerage cross section with progressively thicker layers of fat, which reduces the effective flow area of the sewerage network cross section. Or due to its low specific gravity it rises to the top of the sewer flow and congeals into a progressively larger floating mass (known as a 'fatberg'). In both cases of sewerage cross section coating and floating mass it increases the chance of blockage, due to either solid waste becoming trapped in a reduced cross section or the floating mass becoming a solid plug.

Foreign Objects is a generic descriptor for solid waste found in the drainage network. By definition of being solid, it will not behave in a fluid fashion and will cause flow impedance whenever it becomes trapped.

It must be noted that Figure 4.11 is of storm water culvert and not a sewer culvert. However, the sources of this solid matter will be similar for both sewer and storm water network.

Rags refers to fabric material such as clothing, sanitary pads and nappies. The problem with rags is that they tend to catch on any imperfections in the sewerage network, thereby causing exacerbation of imperfection and increased probability of consequent catching of other foreign objects.

Roots refers to the intrusion of plant life into the sewerage network. Roots are likely to intrude into the sewerage network due to the high nutrient content of the sewer flow, which is attractive to plant life.

Sand is a generic descriptor for blockages caused by soil ingress and will be thoroughly explained in a later section of this report.

Figure 4.11 Examples of Solid Matter to be found in Culverts

Sewer Pump Station: Overflow is the malfunctioning of a sewer pump station either through pump breakdown or blockage.

Sewer: Blocked/Overflow is numerically shown below as the generic descriptor for most combinations of the above blockage causes.

Sewer: Pipe Broken will have much the same causes as collapse as discussed earlier.

All of these causes can occur by themselves or in combination, due to the complex nature of material passing through a sewerage network. Given in Table 4.11 are the listed causes and their recorded number from both the light filtered and heavy filtered data-sets.

It can be clearly seen that the overwhelming majority of blockages are attributed to Sewer: Blocked/Overflow, which lends credence to the observation that municipal workers are likely applying the minimum effort course of using the most generic of descriptors. While this is of no particular concern for the purposes of this research, it does not allow for accurate

Table 4.11 Recorded Blockage Causes

	Light Filter		Heavy Filter	
Reported Failure	**Count**	**Percentage**	**Count**	**Percentage**
Cement Layers	28	0.003	21	0.002
Collapse	200	0.019	148	0.017
Fats	5	0.000	4	0.000
Foreign Objects	26	0.002	21	0.002
Rags	3	0.000	2	0.000
Roots	50	0.005	37	0.004
Sand	11	0.001	3	0.000
Sewer Pump Station: Overflow	14	0.001	4	0.000
Sewer: Blocked/Overflow	1057825	98.863	863350	99.084
Sewer: Pipe Broken	11826	1.105	7740	0.888
Total	1069988		871330	

representation and analysis of what is causing sewer blockages.

Most blockages are either cleared through pressurised hosing, poling (ramming a pole through a blockage) or bucket draglines. The blockages themselves are rarely seen and therefore a more in-depth description is impractical without explicit request.

4.2.5 Seasonal Blockage Patterns

Two representations of the ten-year period blockage patterns are given in Figures 4.12 and 4.13. Figure 4.12 shows the ten-year period with concurrent blockage patterns rising in an ever increasing cycle. Figure 4.13 shows the period but with the blockage patterns plotted in two-time dimensions (by month and year). This serves to create a surface, presenting a clear understanding of the blockage number peaks coinciding with the wet months.

Of interest are the clearly discernible temporal, blockage event peaks coinciding with the wet month periods and blockage event troughs coinciding with the dry month periods. This again, points to the process of blockage creation being exacerbated, during low flow periods,

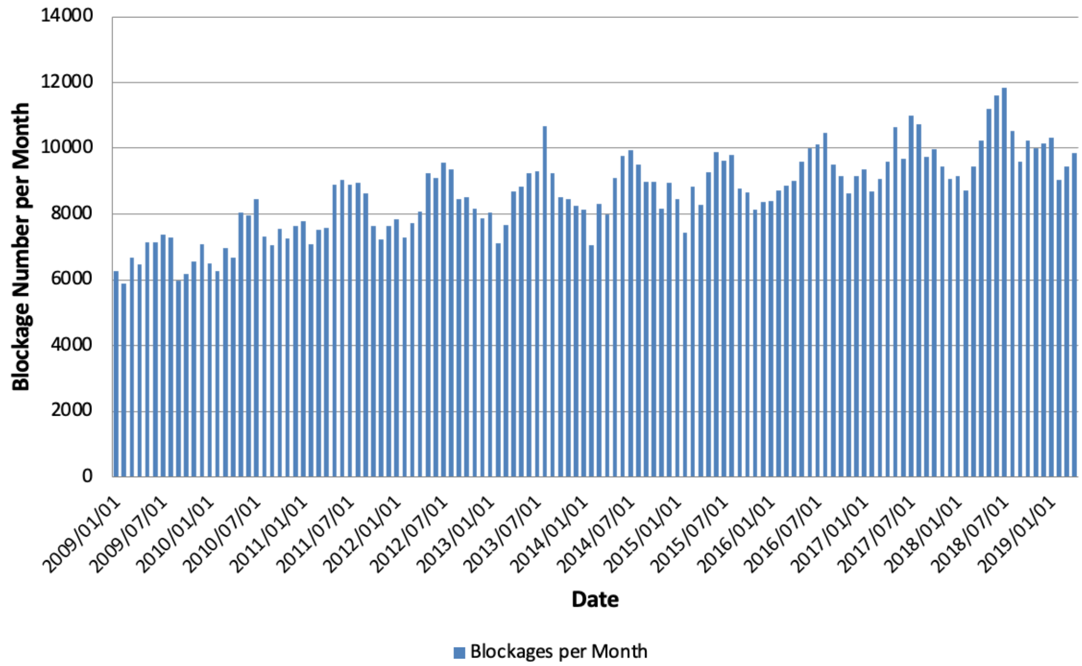

Figure 4.12 Concurrent Blockage Patterns 2009 - 2019

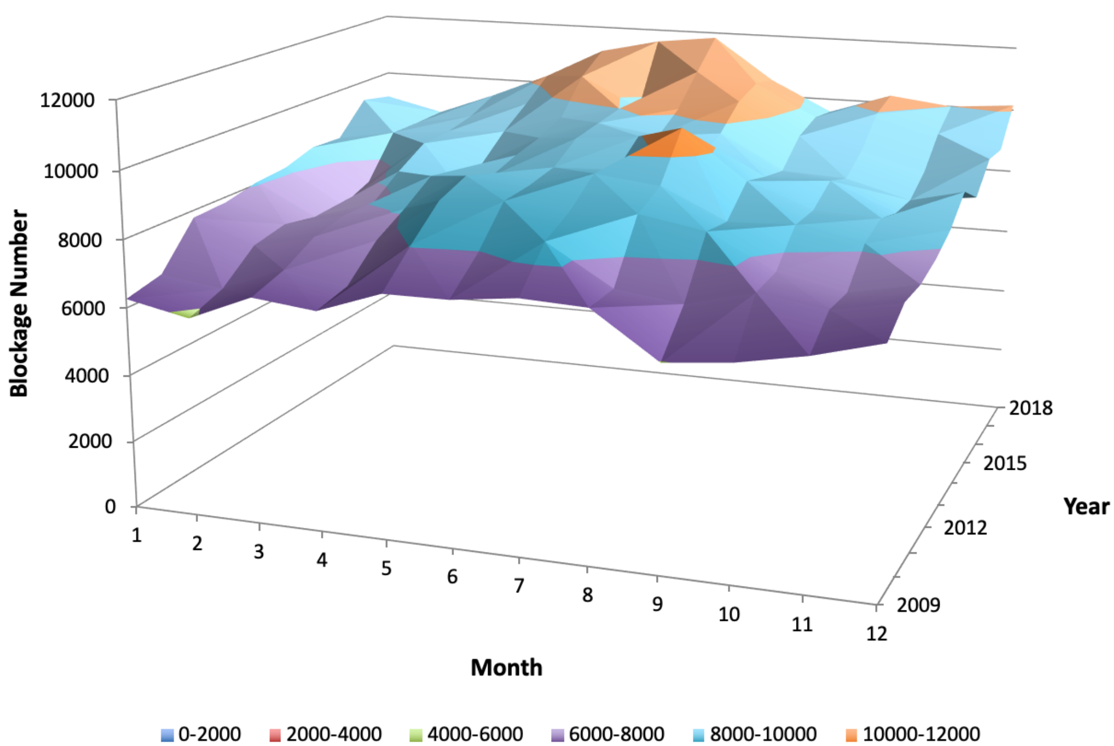

Figure 4.13 Parallel Blockage Pattern Surface 2009 - 2018

when the energy passing through the drainage network in the form of wastewater flow is low and allows for the settling out of sediments and lodging of solid matter.

The corresponding peaks in the wet month periods points to the increased level of discovery of blockages due to a higher flow volume passing through the drainage network, which in turn places the hydraulic constraints of the drainage network under pressure and any blockages will lead to localised drainage network failures, such as flooding and manhole lid displacement. These are then reported and recorded.

Figure 4.14 shows a representation of the blockage patterns for the full time period for both the light filtered and heavy filtered datasets respectively. Furthermore, the graphic was constructed with a 48-month extension to the trendlines displayed to provide an approximation of the future blockage numbers to be expected from the perpetuation of this pattern.

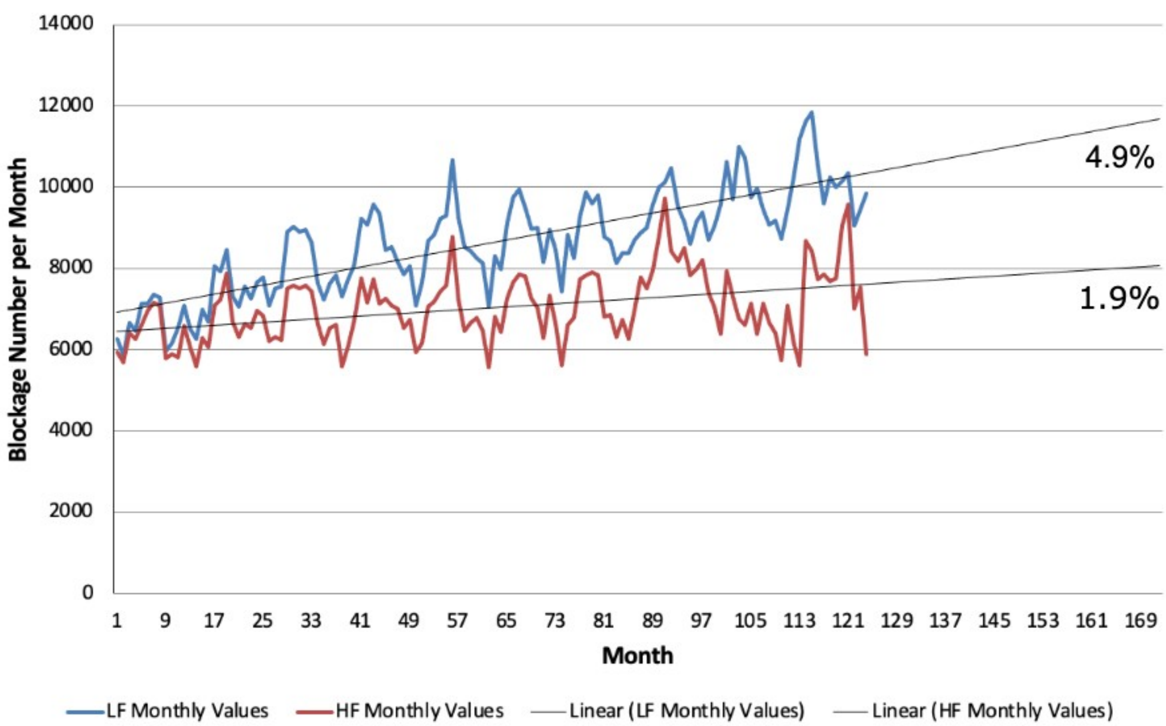

Figure 4.14 Light Filtered and Heavy Filtered Extrapolated Blockage Pattern

The average annual growth rate for the light filtered and heavy filtered data-sets is 4.9% and 1.9% respectively. This can be seen on the right hand side of Figure 4.14.

Of further interest, is that no matter the time of year, there is always a relatively high blockage frequency. This points to a continuous base rate of blockage formation in the network with peaks and troughs varying relative to this base rate.

The addition of the extended trendlines was to create an envelope for probable future blockages numbers. The high and low boundary trendline values, at month 169 correspond to 11 585 and 8 039 blockages per month, for the light filtered and heavy filtered data-sets respectively. These values would be representative of the average monthly value for that year. As such the light and heavy filtered trendlines would imply that in that year the number of blockages experienced in Cape Town would be some 2.25 and 1.55 times greater than Johannesburg's blockage rate. Furthermore it would imply that the average rate of blockages per kilometre per year would in Cape Town would be between 48.6 and 33.7 times greater than the international average.

4.2.6 Areal Blockage Patterns

In Chapter 3, attention was drawn to the decision to display Atlantis, despite it being an areal outlier. Shown in Figures 4.15 are the top 20 problem areas organised by the 'city' descriptor, in descending order of average blockage numbers per day. It can be seen that Atlantis ranks at number 12, with a corresponding daily average of 4 blockages. Therefore, it was considered justifiable to maintain its place in the graphics of Chapter 3. Figure 4.16 displays these problem areas plotted geographically,

It is of particular interest that, almost all of these areas are located in the flatland regions of greater Cape Town. With recollection of the lithological map shown in Chapter 3 (Figure

3.8) these areas correspond to deep sediment beds.

When considering that soil ingress has two likely modes, that of groundwater infiltration (through damaged or ill-fitting pipes and damaged surface appurtenances) and through direct sediment dumping (construction material). The minimum and maximum values of groundwater infiltration, are respectively 0.03 and 0.04 *L/min/m pipe/ m diameter* (Department of Human Settlements, 2019). This corresponds with the supplied estimation of infiltration used by GLS Consulting; however, in the Cape Flats drainage network, GLS Consulting has defined an infiltration rate of 0.08 *L/min/m pipe/m diameter* (double the DHS maximum value).

Consequently, there is a strong case to be made that; there is a high level of movement of sedimentary load from the sediment beds, which dominate the surface geology of Cape Town, into the Drainage Nework.

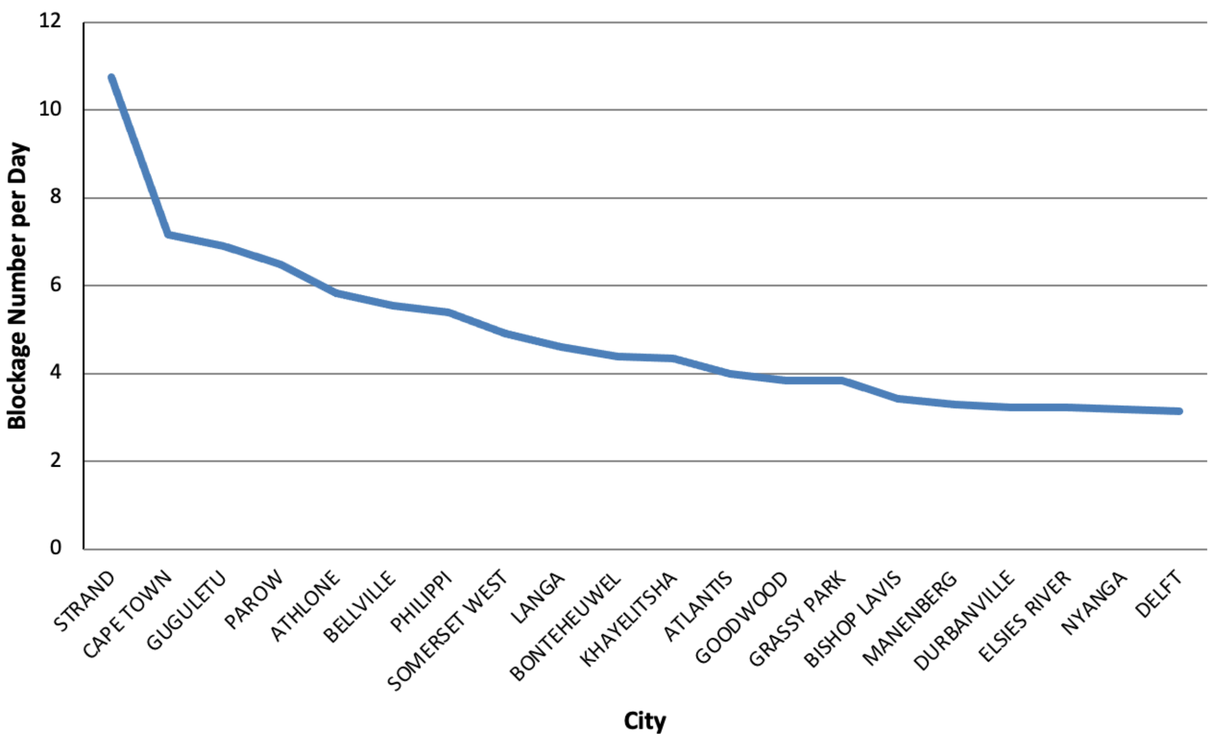

Figure 4.15 Average Daily Blockage Values for Top 20 City Descriptors

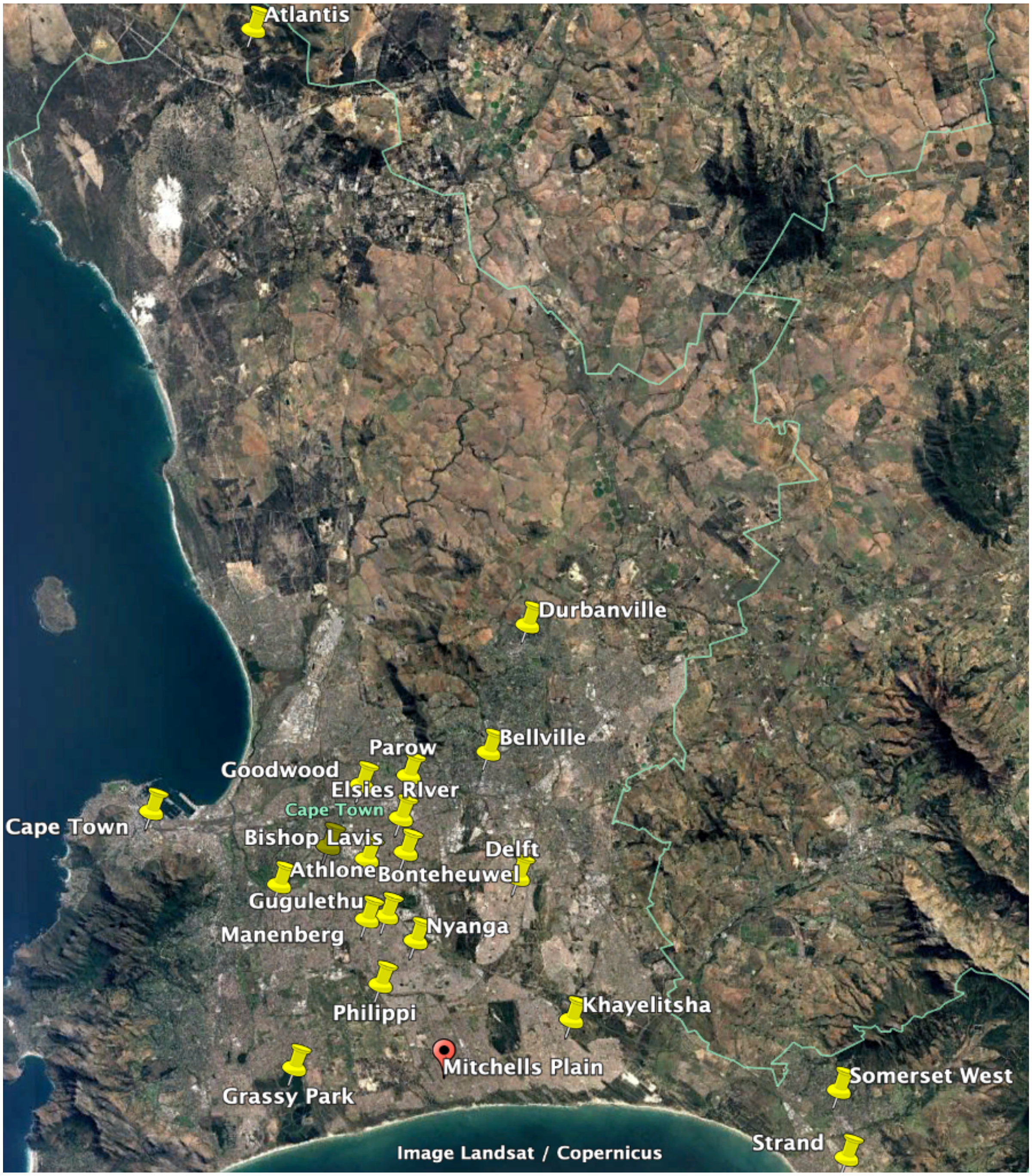

Figure 4.16 Geographical Plot of Top 20 most Blockage Prone Areas (Google, 2019)

Table 4.12 Light Filtered Blockages per Kilometer Drainage Network per Year

Year	Total Blockages	Blockages per *km*
2009	79940	8.4
2010	87710	9.2
2011	96847	10.2
2012	101200	10.6
2013	103992	10.9
2014	104855	11.0
2015	105480	11.1
2016	111554	11.7
2017	117010	12.3
2018	122706	12.9

This information in conjunction with the earlier provided sand trap information, strongly contradicts the impression given by Table 4.11 that sand only caused 11 blockages in the ten-year period. Consequently, it was decided that Chapter 5 should be dedicated to understanding the modes of sediment movement in a drainage network.

According to the GLS Consulting's Masterplan for the City of Cape Town there is a total of 9 538.9 km of sewerage infrastructure in the drainage network. Given in Table 4.12 is the average number of blockages per year per kilometer of drainage network.

This means that should one choose any arbitrary section of drainage network and walk 1 km along it; one should encounter approximately 1 blockage in any given month of the year.

The natural conclusion to this line of investigation is, how much does blockage clearance cost the City of Cape Town per annum? Upon request, the City provided a total figure of R 368 708 421.35 for the year of 2019 for reticulation network maintenance and upgrades. Of this it was confirmed that R 270 million was spent on blockage clearances alone.

What does this figure mean in the greater scheme of South African sanitation systems though? Table 4.13 breaks down the national budget for water and wastewater infrastructure for the years between 2015 and 2018.

Table 4.13 National Budget for Water and Wastewater Infrastructure (DWS, 2017)

	Audited Outcome (ZAR Millions)		
	2015/16	**2016/17**	**2017/18**
Water Resource and Bulk Infrastructure	7230	7718	7649
Regional and Local Water and Sanitation Services	3829	3507	4195
Total	11059	11225	11844

The average audited outcome for these three years is then R 3 844 million. So blockage clearance in Cape Town alone would account for 7% of the National Budget.

Once again it must be pointed out that the city of Cape Town experienced almost exactly twice the number of sewer blockages as the city of Johannesburg. While blockage information was sought for other cities in South Africa none was found. The only other point of reference was found in (Stephenson and Barta, 2005) where it is stated that the international average blockage rate is 0.3 blockages per kilometer per annum. Therefore Cape Town has a blockage rate almost 40 times higher than the international average.

With recollection of the fact that Cape Town has the best performing sanitation system in the country, the inference is that all other sanitation systems in South Africa should be spending proportionately more than Cape Town on blockage clearance. In the event that the other sanitation systems are unable to foot the bill of blockage clearance then the implication is that the respective drainage networks of the sanitation systems will be unable to function efficiently.

The repercussions of this will be improvisation of wastewater disposal. What this means in reality is that any wastewater, which cannot be processed due to drainage network blockages will simply find its way into the stormwater system, from where it will find its way into the nearest stream or river, via which it will flow into a pond, dam or the ocean. The resultant costs to communities, which these sanitation systems are supposed to be servicing, will be high, either financially or physically. Inverting the cost benefit ratio of 2:11 given in the Introduction, mismanagement of wastewater could conceivably result in a R5.50 expense for every R1, which is mismanaged.

Chapter Five

Sedimentation

5.1 Introduction

In the previous chapter, a blockage analysis for the city of Cape Town was conducted. A number of blockage causes were listed. While most of these blockage causes have very little literary research and modelling, sedimentation has had extensive, international efforts into research and modelling.

This, in combination with the aforementioned sandy nature of much of Cape Town's terrain steered the focus of this research towards creating a theoretical understanding of the mechanisms of sedimentation in sanitation systems. It must be noted that, due to extremely limited data being available for sedimentation patterns in South Africa's sanitation systems, there exists great uncertainty about the vivirithmic nature of sedimentation.

Attention is once again drawn to the report of some 8 136 tons of sediment being removed annually from the sand traps of two of the four reticulation regions in the city of Cape Town. Unfortunately the other two regions did not reply to the request for information. So an accurate total for the city is not known.

Of further interest from the request for information were the general complaints of vandalism of sand traps. Where the lids were regularly removed and the traps were used as dumping locations for trash. This serves to significantly decrease the capacity of the sand traps to function properly as well as increasing cost of maintenance due to more regular cleanings being required.

With the aging and material deterioration of the reticulation network in conjunction with increasing reports of vandalism of chamber lids (manhole and sand trap), more and more sediment will find its way into South Africa sewer systems. Consequently there will be an increasing number of sediment related blockages.

This increasing intrusion of foreign matter (sand and trash0 into the Cape Town drainage network provides good grounds for understanding why Cape Town has an average blockage rate approximately 40 times greater than the international blockage rate.

The first section of this chapter provides a selected literature review of the theory of sedimentation. The second section provides the set of equations used to calculate the mechanisms of sedimentation. Due to the complex interrelationship of many different parameters, there is high variability and uncertainty of calculated outcomes. Consequently, following on from the provision of the sedimentary equations, an uncertainty analysis was conducted on selected equations.

5.2 Selected Literature Review

Sedimentation in sewerage networks is dependent on many different variables, such as, but not limited to cohesiveness, pipe diameter, flow velocities, shear stresses, sediment sources, flow depths and gradients (Ghani, 1993; May, 1993; Nalluri, 1995; Berlamont and Torfs, 1996; Ebtehaj and Bonakdari, 2013).

Generally, it is acknowledged that sedimentation and its prevention is one of the most important considerations for the functional design of a sewerage network. As such, there tend to be minimum self-cleansing sewerage design standards stipulated by a country or some other regulatory body. These minimum design standards tend to be in the form of minimum slopes, minimum velocities or minimum shear stresses (Nalluri, 1995).

The purpose of these minimum design standards is to ensure that on a regular cycle the sewerage network will perform under such conditions that the scouring of sedimentary deposits is highly probable. The periodicity of this scour is attuned to high flow. However, it has been shown that these design constraints are oversimplified, with the design constraints over designing pipes of less than 500 mm diameter and under designing pipes of over 500 mm diameter (Ghani, 1993; Nalluri, 1995).

Historically, it has been acknowledged that during wet periods the sewerage network experiences higher flow loading and during dry periods the sewerage network experiences lower flow loading. This was proven in Chapter 4, which clearly presented the information that significantly higher flow loads were experienced for the wastewater treatment plants in the wet periods. Considering that all the flows in the wastewater treatment plants would have been routed through the sewerage network, this provides validation of higher flow loading being experienced in the sewerage network during wet periods.

Due to the lower flow loading during dry periods, there is an accordingly lower total energy experienced in the system. Not only does this reduce or entirely prevent scouring of sedimentary deposits, but also allows for the further settling out of sedimentary suspended load onto the pipe invert. With protracted lengths of low flow, this settling out of sedimentary load, may cause significant sedimentary deposits to form.

The issue of cohesiveness is of importance, since its presence or absence can radically alter the efficacy of models. Generally, storm water sewerage networks tend to carry mostly a sand or grit sedimentary load, while foul water sewerage networks carry a combination of sand, grit and a biological sludge load (May, 1993). The sand or grit form of sediment tends to be non-cohesive, therefore the sediment deposits generally do not undergo processes such as cementation. However, with the addition of cohesive materials such as clays or organic solids, cementation and consolidation tend to occur. This, in turn, tends to increase the requisite shear stresses required for scouring of a sedimentary deposit (May, 1993; Berlamont and Torfs, 1996; Ebtehaj and Bonakdari, 2013). However, it has been shown that after the threshold of motion is surpassed for a sedimentary deposit, cohesion no longer plays a significant role in sedimentary behaviour (May, 1993).

The three most significant effects of sedimentation in a sewerage network are the reduction of flow capacity through reduction of flow cross sectional area, increased flow resistance (May, 1993) and the increased level of pollutants in the flow when sedimentary beds are scoured and resuspended (May, 1993; Berlamont and Torfs, 1996; Ebtehaj and Bonakdari, 2013). It has been shown that sediment deposit scour can cause an additional 30 to 80% of pollutant mass in wet weather flows (Oms *et al.*, 2008).

5.3 Sedimentation Equations

It is generally accepted that there are two types of sediment in storm water and wastewater networks, namely, non-cohesive and cohesive. Non-cohesive sedimentation theory is more applicable to storm water networks, due to a generally low organic and clay content. Sanitation systems, on the other hand have relatively high organic content such as grease and tars (Berlamont and Torfs, 1996), therefore they experience cohesive sedimentation.

Due to the sandy surface geology of Cape Town the nature of intruding sediments will be coarse grained, non-cohesive material. However, with the addition of organic materials the sediment undergoes agglutination and behaves in a cohesive fashion.

Consequently, this chapter will only focus on the theory of cohesive sedimentation. The following equations were primarily adapted from the literature of (Mays, 2001c).

5.3.1 Boundary Shear Stress

$$\tau_0 = \frac{f}{8}\rho V^2 \qquad [N/m^2] \tag{5.1}$$

Where:

ρ = fluid density $[kg/m^3]$

f = Darcy-Weisbach friction factor

V = mean flow velocity $[m/s]$

5.3.2 Darcy-Weisbach Friction Factor

$$\sqrt{f} = \frac{1}{-2log(\frac{k}{3.7D} + \frac{2.51\nu}{D\sqrt{2gDS}})} \qquad [Dimensionless] \tag{5.2}$$

Where:

k = absolute roughness $[m]$

D = diameter $[m]$

ν = kinematic viscosity of fluid $[m^2/s]$

g = gravitational constant $[m/s^2]$

S = slope $[m/m]$

5.3.3 Shear Velocity

$$U^* = \sqrt{\frac{\tau_0}{\rho}} \qquad [m/s] \tag{5.3}$$

5.3.4 Fall Velocity 1

$$w = \frac{[9\nu^2 + 10^9 d_{50}^2 g(s-1)(0.03869 + 0.02486 d_{50}) - 3\nu]^{0.5}}{(0.11607 + 0.074405 d_{50})10^{-3}} \qquad [m/s] \tag{5.4}$$

Where:

d_{50} = median particle size $[mm]$

s = specific gravity $[dimensionless]$

5.3.5 Fall Velocity 2 (Sadat-Helbar *et al.*, 2009)

$$D_* \leq 10 : w = 0.033 \frac{\nu}{d_{50}} [\frac{d_{50}^3 g(s-1)}{\nu^2}]^{0.963} \qquad [m/s] \tag{5.5}$$

$$D_* > 10 : w = 0.51 \frac{\nu}{d_{50}} [\frac{d_{50}^3 g(s-1)}{\nu^2}]^{0.553} \qquad [m/s] \tag{5.6}$$

Where:

D_* = Effective Particle Diameter

Note that in this equation d_{50} is in m and not mm.

5.3.6 Effective Particle Diameter

$$D_* = [\frac{g(s-1)}{\nu^2}]^{1/3} d_{50} \tag{5.7}$$

Note that in this equation d_{50} is in m and not mm.

5.3.7 Extended Shields Critical Shear Stress (Cohesive Sediment)

$$\frac{\tau_0}{g(\rho_s - \rho)d_{50}} = \tau_c^* + \frac{\tau_s}{g(\rho_s - \rho)d_{50}} \tag{5.8}$$

Where:

ρ_s = sediment bulk density $[kg/m^3]$

τ_c^* = Shields critical shear stress (obtained from the Shield's diagram) $[N/m^2]$

τ_s = Sediment bed shear strength $[N/m^2]$

This equation is used to check the threshold value of incipient motion. If the left-hand side is greater than the right-hand side, sediment movement will occur. If not, the sediment will remain fixed. To access the Shield's diagram, the reader is directed to either of the resources of (Mays, 2001c) or (Chadwick, Morfett and Borthwick, 2013).

5.3.8 Minimum Velocity (U.S. Design Criteria – Camp Formula)

$$V_m = \sqrt{\frac{(8Kg(s-1)d_{50})}{f}} \qquad [m/s] \tag{5.9}$$

Where:

K = cohesiveness constant

5.3.9 Minimum Slope (U.S. Design Criteria)

$$S_m = \frac{(fV_m^2)}{4R2g} \tag{5.10}$$

Where:

R = Hydraulic Radius = Flow Area/Wetted Perimeter $[m]$

5.3.10 Minimum Boundary Shear Stress (U.S. Design Criteria)

$$\tau_m = \rho g K(s-1)d_{50} \qquad [N/m2] \tag{5.11}$$

5.3.11 Erosion Rate

$$E = M\frac{(\tau_0 - \tau_s)}{\tau_s} \qquad [kg/m2] \tag{5.12}$$

Where:

M = rate constant $[kg/m^2]$

5.4 Uncertainty Analysis

In light of the limited data available for localised conditions within a sewer system, it is difficult to accurately apply the equations above. Consequently, it was decided that a general investigation of the spaces of the equations should be conducted. This investigation serves two purposes, to show the variation of outcome and subsequently to provide an uncertainty analysis.

Due to the large variability of parameters, it was necessary to code the equations in Java (available in Appendix B). The Java code was then populated with ranges of parameters over which the equations could be varied.

It must be noted further, that due to each equation having numerous different parameters, and some parameters themselves being functions of other parameters, the solution spaces become very large, very quickly.

Take for example the boundary shear stress equation, which has 3 parameters. This means that the solution space will exist in 3 dimensions. Should it be decided that each of these 3 parameters be defined over a range of 8 distinct values, then the solution space will be defined by a 3-dimensional matrix, with each dimension having a size of 8. For representation, graphics are generated for each of these dimensions. In numerical data format, each of these graphics will be an 8 by 8 table and there will be 8 tables. This means there are 512 data points generated. For a 4-parameter equation, the solution space will be in 4 dimensions, a 5-parameter equation will be in 5 dimensions and so on.

Table 5.1 presents the chosen parameter variation ranges. Note that the intervals between

Table 5.1 Chosen Parameter Variations

V (m/s)	tau$_0$ (N/m^2)	d$_{50}$ (mm)	K	D (m)	S	k (mm)	tau$_s$ (N/m^2)	s	v (m^2/s)	M (kg/m^2)
0.3	0.2	0.01	0.04	0.11	8.33E-03	0.03	0	1.01	1.79E-06	1.00E-03
0.5	0.4	0.02	0.08	0.16	5.00E-03	0.06	1	1.1	1.31E-06	1.00E-02
0.6	0.6	0.03	0.12	0.2	4.00E-03	0.1	2	1.4	1.14E-06	1.00E-01
0.65	1	0.04	0.16	0.25	2.86E-03	0.2	3	1.6	8.90E-07	
0.7	1.4	0.05	0.2	0.315	2.00E-03	0.3	4	2	8.00E-07	
0.8	1.8	0.06	0.24	0.355	1.66E-03	0.4	5	2.3	6.60E-07	
1	2.2	0.08	0.28	0.45	1.43E-03	0.5	6	2.5		
1.2	2.6	0.09	0.32	0.525	1.25E-03	0.6	7	2.6		
1.4	3	0.1	0.36	0.6	9.09E-04	0.7	8	2.7		
1.6	3.4	0.2	0.4	0.675	7.69E-04	0.8	9			
1.8	3.8	0.3	0.44	0.75	6.66E-04	0.9	10			
2	4.2	0.4	0.48	0.825	5.55E-04	1				
2.5	4.6	0.5	0.52	0.9	5.00E-04	1.1				
3	5	0.6	0.56	1.05	4.35E-04	1.2				
3.5	6	0.7	0.6	1.2	3.57E-04	1.3				
4	7	0.8	0.64	1.35	2.94E-04	1.4				
4.5	8	0.9	0.68	1.5	2.50E-04	6				
5	9	1	0.72	1.65	2.17E-04					
5.5	10	1.2	0.76	1.8	1.89E-04					
		1.4	0.8							
		1.6								
		1.8								
		2								

parameters are not always constant. Each of the parameter ranges was chosen either to correspond to information taken from literature, which were defined as the most commonly found in sewers. Other parameter ranges were chosen to fully encompass likely scenarios to be found in sewers. A summary of whether each parameter was chosen or taken from literature is presented below.

- V – assumed range
- τ_0 - assumed range
- d_{50} - assumed range extended from information found in (Mays, 2001c)
- K - range taken from (Mays, 2001c)
- D – based on values taken from (Department of Human Settlements, 2019)
- S – based on values taken from (Department of Human Settlements, 2019)
- k – based on combination of information taken from Mays (2001c) and Chadwick, Morfett and Borthwick (2013)
- τ_s - assumed range
- s – based on values from Mays (2001c)
- ν - based on values from (Engineeringtoolbox.com, 2015)
- M – Based on values from Mays (2001c)

In the case of parameters not shown in Table 5.1, the values were implicitly calculated in the Java code on an ad hoc basis, corresponding to the respective parameters shown in the table.

Due to these parameter variations generating an amount of data, too large to reasonably be displayed in this research, only a select few graphics are displayed for each of the equations. The order of graphical presentation will follow that of the equations presented above.

Clarification is provided of the following parameter choices. The last parameter of the k-parameter set, is based on information in (Chadwick, Morfett and Borthwick, 2013) stating that, a k value of 6 mm is suitable for a sewer with severe sliming and sedimentary bed.

Furthermore, the D and S values taken from (Department of Human Settlements, 2019) are based on South African minimum design constraints to maintain self-cleansing velocities. Consequently, all calculations with either of these parameters by default, is also calculated with corresponding parameter pairing (example given diameter-slope pairing 1, refers to a pipe with a diameter of 110 mm and slope of 8.33E-3). The kinematic viscosity range corresponds to temperature of 0, 10, 20, 25, 30 and 40 ° Celsius.

Finally, the range of median particle size includes two values, 0.2 and 0.6 mm. These values respectively represent Witsands and Sprinfontein soils, which are two of the superdominant soil types in the Cape Town region (Allsopp, 2017).

For each of the following figures, the parameter variation in two dimensions is presented on the x and y-axes, and the calculated output on the z axis. The fixed parameters will be explicitly stated.

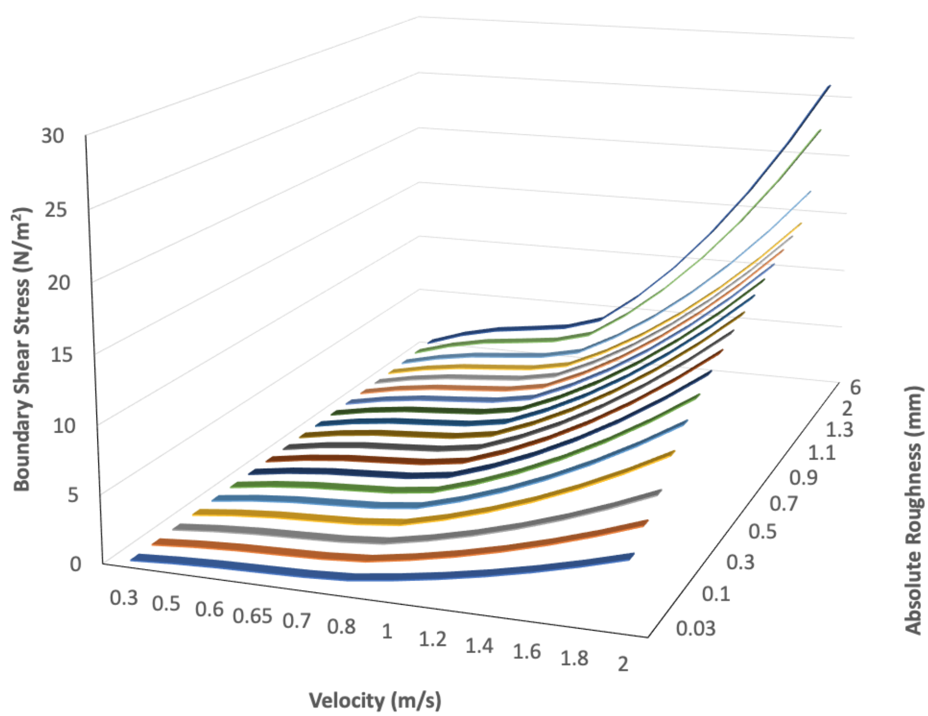

Figure 5.1 Variation of Boundary Shear Stress with Friction Factor and Velocity

Through fixing parameters; $k = 0.03\ mm$ (new pipe roughness) (Chadwick, Morfett and Borthwick, 2013), $\nu = 1.14\text{E-}6\ m2/s$ and $\rho = 1000\ kg/m^3$, Figure 5.1 was generated. It can be seen that both the variation along the x and y axes have a parabolic curve form. This dual parabolic form is due to the boundary shear stress and friction factor being a function, $f(g^2)$, where g is the respective internal component for the two functions, displayed before.

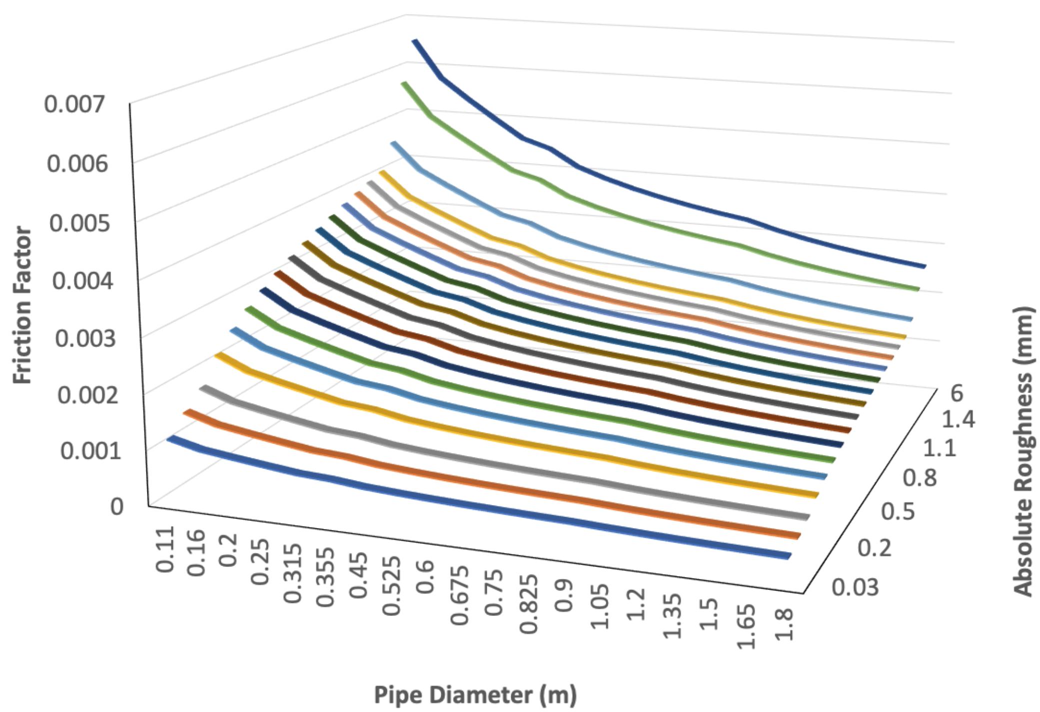

Figure 5.2 Darcy-Weisbach Friction Factor Uncapped

Figure 5.3 displays the fall velocity equation (Fall Velocity 1) as supplied by (Mays, 2001c). It is evident that the values generated by this function, are many orders of magnitude greater than is reasonable for the fall velocity of a sediment particle.

Subsequently, an alternative fall velocity function was chosen from the literature source (Sadat-Helbar *et al.*, 2009). The solution space of this equation is displayed in Figure 5.4. Of interest to note in this fall velocity graphic is the break line as the effective particle diameter becomes greater than 10.

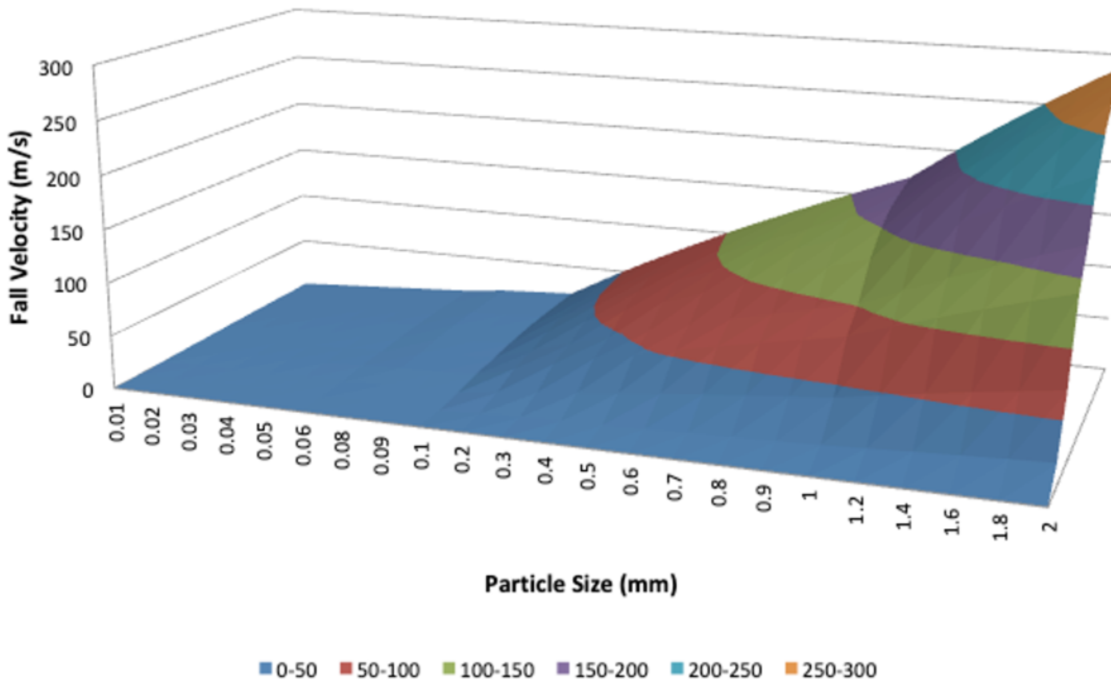

Figure 5.3 Fall Velocity 1

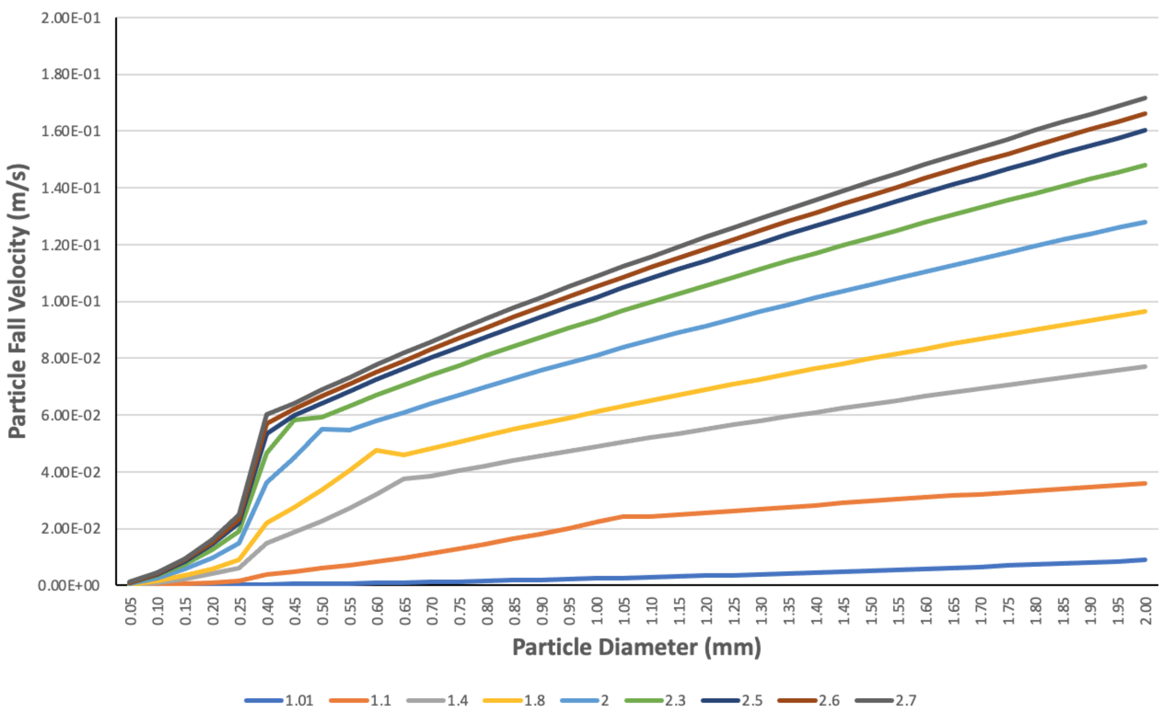

Figure 5.4 Fall Velocity 2 (Sadat-Helbar *et al.*, 2009)

The results of Equations 5.5 and 5.6 are far more reasonable, and these equations were therefore considered to be more reliable.

The curves displayed in Figures 5.5 and 5.6, are for a small pipe (displayed in blue with fixed parameters, $D = 0.16\ m$, $k = 0.06\ mm$, $S = 0.05$, $s = 1.01$, $d_{50} = 0.2\ mm$ and $f = 0.0478$ – corresponding to the conditions of a Witsands soil type) and a large pipe (displayed in red with fixed parameters, $D = 0.6\ m$, $k = 0.06\ mm$, $S = 0.05$, $s = 2.7$, $d_{50} = 0.6\ mm$ and $f = 0.0192$ – corresponding to the conditions of a Springfontein soil type).

Note that the red curve is plotted against the right-hand axis. It can be seen that the minimum requisite velocity ranges from 13 – 57.8 m/s. This is again an impractically large value.

The magnitudes of these values were deduced to stem from the chosen set of parameters being the maximum in their respective ranges. When substituted into the equation this caused for a strong compounding of an equation, which already lends itself to high values. This brought the validity of the equation into question, since these high parameter values are quite possible in a sewer. Whereas, the generated results of the equation are entirely impractical.

The minimum slope solutions spaces, displayed in Figure 5.6, were again calculated for a small and large pipe with sediment beds. Consequently, the fixed parameters are the same as those for the minimum velocity. These respectively correspond to the Witsands and Springfontein soil types.

It can be clearly seen in Figure 5.6, that the larger the pipe diameter, the smaller the requisite slope is. This corresponds to the design philosophy of simple sewer design as formulated by (Bakalian *et al.*, 1994). This reduction of requisite slope leads to reduced excavation costs. However, a cost balance analysis should be conducted during design, to find the optimum

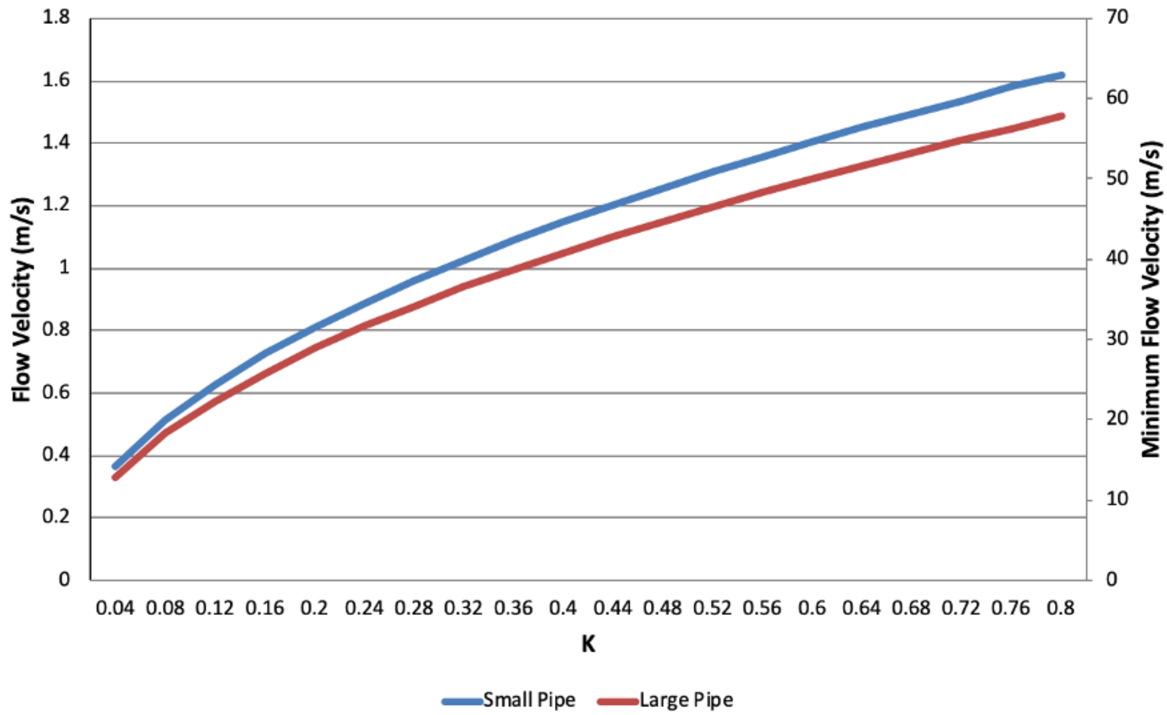

Figure 5.5 Minimum Flow Velocity Solution Curves for Small and Large Pipe

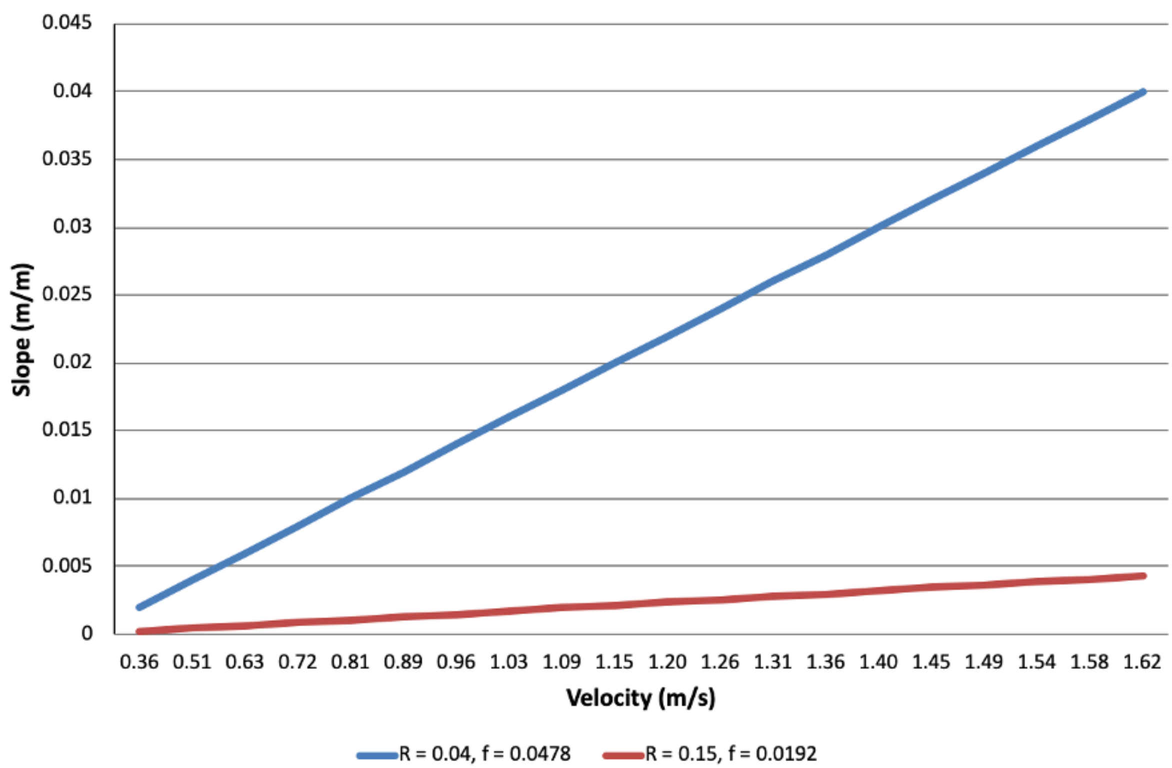

Figure 5.6 Minimum Slope Solution Curves

cost savings based on excavation versus pipe costs.

In Figure 5.7, the solution space shows that the higher the density of the particle, the higher the requisite boundary shear stress will be to ensure self-cleansing. The fixed parameter for this solution space is $d_{50} = 0.01\ mm$. The variation of $K = 0.04$ through $K = 0.8$, are stated as the corresponding values, to the Shield's criterion for fine sands and to ensure self-cleansing, respectively (Mays, 2001c).

It must be noted that the outputs generated for $K = 0.8$ are all impractically large, despite being the recommended value to ensure self-cleansing. Therefore, it is recommended that a value of $K = 0.24$ is used, since this leads to more reasonable minimum shear stresses.

Figure 5.8 shows the rate of erosion with the variation of bed shear stress (boundary stress) and bed shear strength. It can be clearly seen that the lower the shear strength of the sediment bed, the greater the rate of erosion and vice versa, which intuitively seems reasonable. The fixed variable was M = 1E-4 kg/m^2.

A final point of interest is Equation 5.3. This equation is a ratio of the boundary shear stress and the fluid density and was used by Shield's to define the relationships between flow velocities and entrainment of particles which he would then go on to publish as the Shield's diagram (Chadwick, Morfett and Borthwick, 2013). This Shield's diagram has for a long time been the standard manner in which to assess sediment entrainment in sewers and other sediment carrying channels.

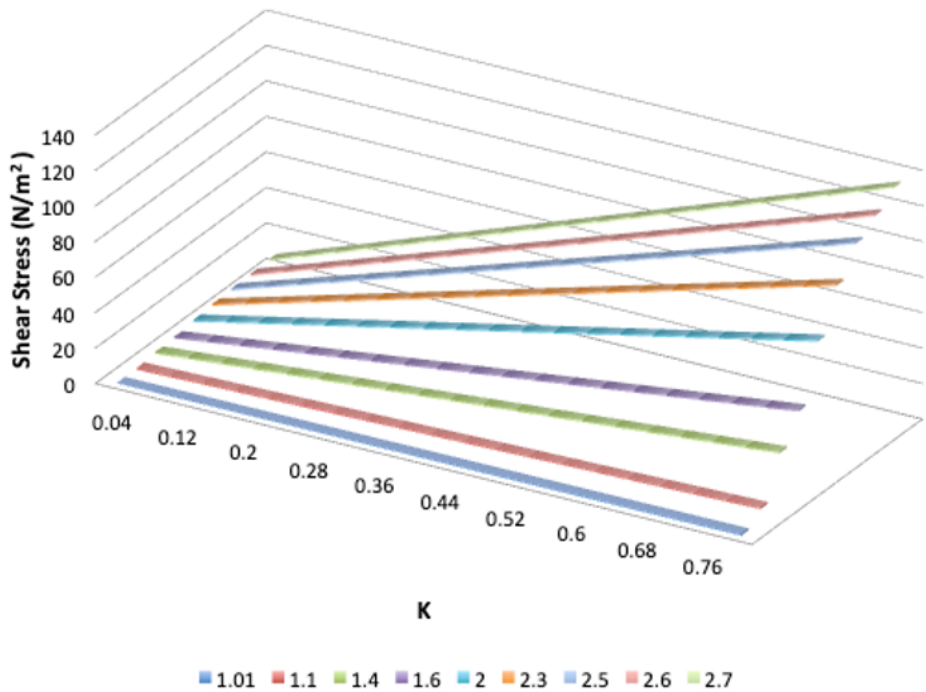

Figure 5.7 Minimum Boundary Shear Stress Solution Curves

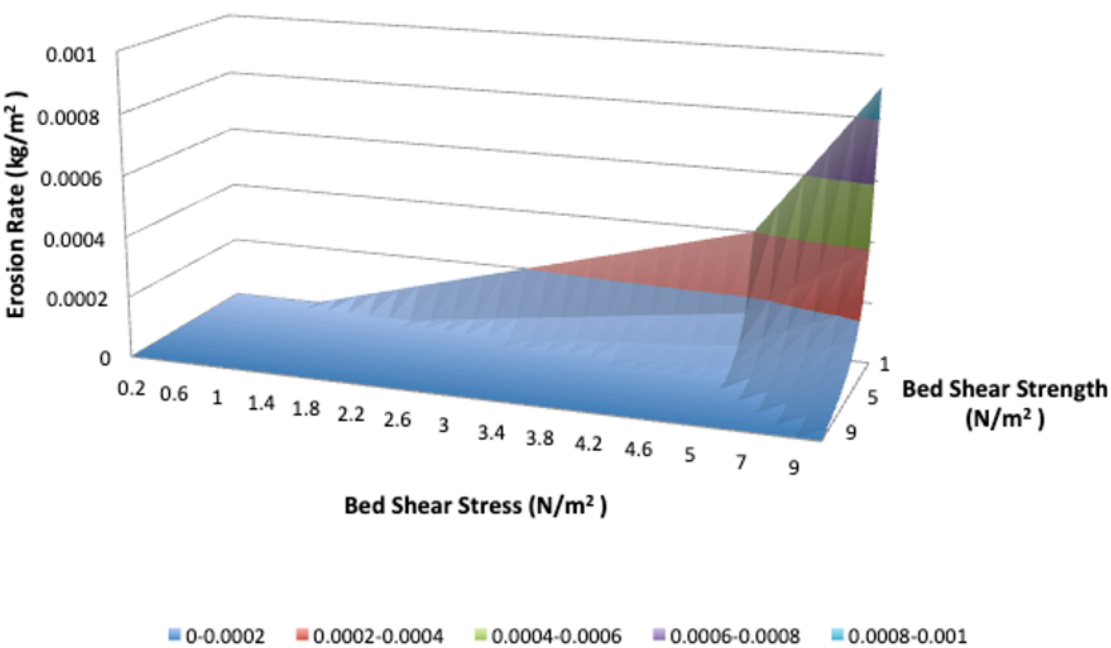

Figure 5.8 Erosion Rate with Variation of Bed Shear Strength and Stress

5.5 Discussion

5.5.1 Function Lacunas

By defining entire ranges of probable parameter variations for South African sewer networks, this uncertainty analysis created understanding of the form equations take over uncertain domains.

Further faults were found to exist in the sediment particle Fall Velocity 1 and the U.S. Minimum Design Criteria. With reasonable parameter inputs, completely implausible results could be generated.

While it would have been possible simply to exclude these equations from this research, it was decided that since they are found in a credible literature resource, they should be openly questioned and not discarded. What they serve to show is one of two things. Either they are misrepresented in text by misunderstanding of the author (Mays, 2001c), or they have been incorrectly recorded in the text.

Again, these equations could have been omitted from this research but that would have been counterproductive. The purpose of this section of research is to understand the theoretical mechanics of sedimentation. One must be able to see when a solution is either plausible or implausible. Omission due to fault in science is dangerous. For it is tantamount to lying by omission. It is better to see the fault, admit its existence and if possible to correct it. For the purposes of this research an investigation and calibration of these equations was deemed beyond the decided scope.

Of further interest is the erosion rate solution surface. In the Selected Literature Review at the beginning of this chapter, it was pointed out that after the threshold of motion is surpassed for a deposit, cohesion no longer plays a significant role in the behaviour of the deposit. The implication of this, is that the erosion rate surface should have a step discontinuity in it corresponding to the threshold of sediment motion. Therein after the erosion rate should increase markedly.

5.5.2 Interpretation

Boundary Shear Stress

The boundary shear stress is the single most important variable in maintaining self-cleansing of a sewer pipe. It is the basis of both standard sewer design (e.g. boundary shear stress as a function of minimum flow velocity) and simple sewer design (e.g. where the design constraint is placed on having a minimum tractive force of 1 N/m^2).

The boundary shear stress is a function of three variables. Pipe friction, fluid density and flow velocity. Pipe friction varies with age, material degradation, sliming, sediment bed formation, pipe diameter and slope, and fluid viscosity.

As the pipe develops more impeding features (as the pipe ages) it will increase the friction factor which will in turn increase the boundary shear stress. This in turn serves to decrease the flow velocity of a fluid under the effects of gravity. The fluid density will remain largely constant.

The flow velocity will vary in accordance with the hydrograph of a given section of sewer. This means that there will be a constantly varying flow velocity as more or less water is sent down the section of sewer. This links into the flow conditions presented in Chapter 2.

Due to the variation in flow rates in accordance with user demand, any given section of a sewer will experience numerous different flow conditions in any given day. This will in turn affect the flow velocities, boundary shear stresses and settling out or entrainment of sediment.

This velocity variation is why there is a design condition of a minimum flow velocity being achieved once a day.

Sediment Fall Velocity

A sediment particle will have a settling velocity in accordance with its specific gravity and its shape factor. Denser, smoother particles have a higher fall velocity. Less dense, larger particles have a lower fall velocity.

What this means in terms of the variability of flow conditions in a sewer is that during low flow more sediment particles are likely to settle onto the pipe invert. During high flow conditions, the sediment particles will be more likely to be carried by the flow.

If a sediment deposit should cause a restriction of pipe cross section and increase friction then the fluid will flow at a lower velocity. This is turn will increase the likelihood of the washload settling out.

Conversely, when a sediment deposit is cleared there will be a decrease in friction and in increase in flow area allowing a higher flow velocity and a minimised likelihood of sediment settling out of flow.

Erosion Rate

The erosion rate is linked to the boundary shear stress and the cohesiveness of the material.

Higher boundary shear stresses are likely to increase rate of erosion. While higher cohesiveness in the sediment will lead to decreases in rates of erosion.

5.5.3 Variability and Uncertainty

The previous section showcased the strengths and weaknesses of sedimentation equations. It is important to understand these equations' bounds of reliability. However, more important still is to understand the underlying phenomena which they are describing mathematically. It is only through an understanding of these principle phenomena that the equations can be safely and accurately applied.

It is difficult for humans to perceive in multiple dimensions all at once. Consequently much of the solutions we create are grounded in reductionist, interlinking 2 dimensional models of the full complexity of a system.

In the case of systems of flow, variability stems from; flowrate, cross sectional area, fluid density and temperature, solids, slope, boundary friction and velocity. Models are instantiated on static 2 dimensional situations, and then incrementally complexity is added until a pseudo-dynamic model is created.

The reality of what is happening in sewers is that the system is pulsing like an intestine. Sending waves and tides of matter and energy down a black box system, in perpetuity. Without constant monitoring of what is actually happening in the sewers in real time, all we truly know are the physical conditions of the sewer at its initiation point in time.

Therein after the complex interactions of the sewage flow cause the system to 'grow'. This happens through such things as the migration of sediment deposits, growth of biofilms, slimes and 'fatbergs'. Pipes break or are displaced. Solid matter is introduced and radically alters cross sections.

The result is that immediately after construction sewers become the civil engineering equivalent of Schrödinger's Cat. A black box system in which it is uncertain whether the system is alive (functioning properly) or dead (no longer functioning). Until the sewer section is actually opened up and looked into, it *cannot* be known what is happening inside.

This is what uncertainty quintessentially is. A dualistic state of not knowing.

There is an argument that Schrödinger created his famous cat experiment as a sort of high-browed academic joke. The reasoning is that around this time in the newly born field of quantum physics, there was a mass analysis paralysis caused by the sheer uncertainty of the field (the basis of Heisenberg's Uncertainty Principle).

So Schrödinger created this thought experiment, where until the box is opened the cat is theoretically both alive and dead. A cat existing in a dualistic state. However, in reality the cat could only be one or the other.

Schrödinger's message was simple. Yes, the universe is infinitely complicated and uncertain. Quantum physiscists could just put down their pens and give up due to not knowing how to proceed with a dualistic problem. Or quantum physicists could acknowledge faults in their systems, overlay the insurmountable uncertainties with simplifying assumptions and proceed as though they were true and see where this led to. Now the world plays host to such wonders as the Large Hadron Collider at CERN where quantum physics can be tested.

What Schrödinger gifted quantum physics was the ability to bullishly ignore confounding uncertainty in order that progress may be made. He created a quantum physics joke about the efficacy of rules of thumb.

The minimum self cleansing velocity of 0.65 m/s is a rule of thumb since it varies significantly according to the regulatory body of a given location in which it is implemented. However, as a rule of thumb it is highly effective if it is maintained.

Sewerage networks are designed to maintain this minimum self-cleansing velocity. Pipes are sized and gradients are chosen. Then the system is built and covered. Therein after it is out of the hands of engineers and becomes the domain of nature's whimsy.

The entelechy of sewer systems the world over shows that they will have external water and sediment intrusion. They will collect solid matter dumped by humans. They will experience flow cross section reduction through biolfilming, sliming, solid matter blockage and sediment bed formation.

These entelechical developments in the system need to be acknowledged and designed for. If they are not, the result *will* be system failure to some degree. This has been proven in Chapter 4. The sanitation system of Cape Town is failing to some degree.

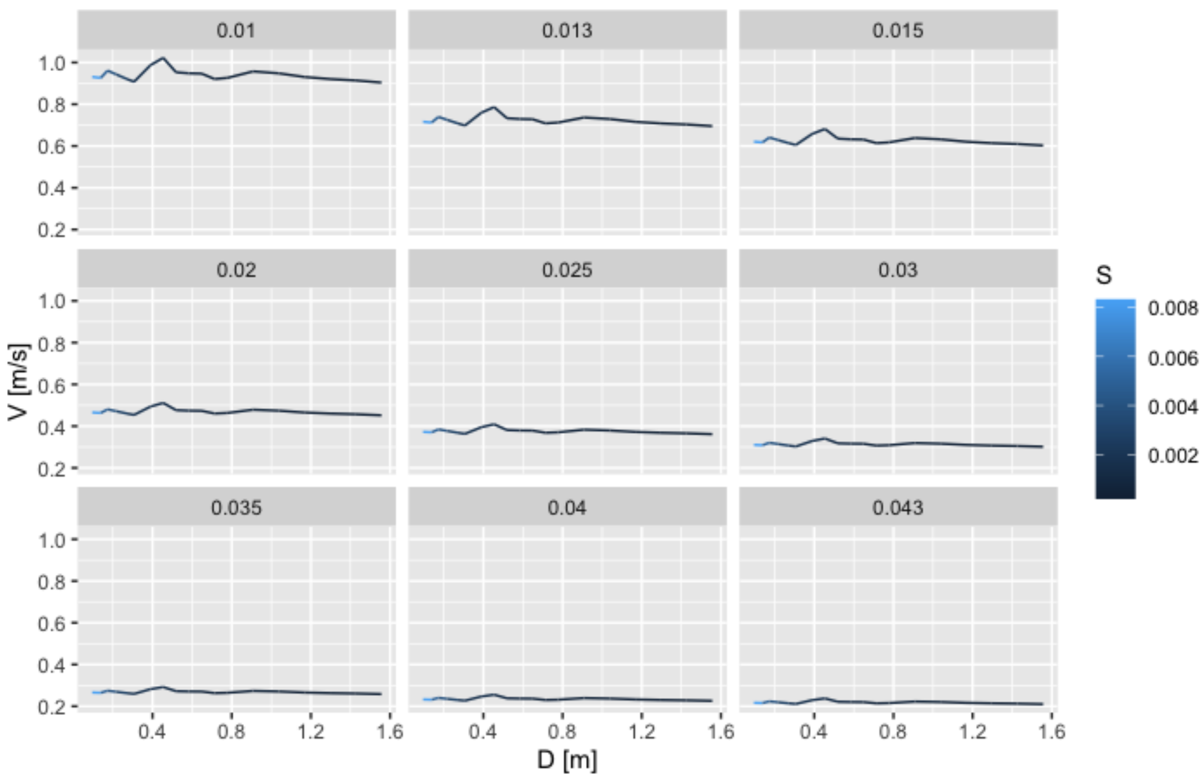

Figure 5.9 Manning's Equation Velocity Variation versus n

The graphs in Figure 5.9 show the variation of velocities against varying Manning's n coefficients. Furthermore, the diameters and slopes were chosen in accordance with the minimum recommended from (Van Zyl and Van Dijk, 2011) to ensure self cleansing. The range of n values was chosen to fit with information from (Van Zyl and Van Dijk, 2011) on the effects of sliming on friction in a pipe. The values generated for these graphs was done with Manning's Equation and a set flow depth of 70%. The code for the generation of the data presented in these graphics is available in Appendix C.

Considering that sewer conduits will experience an increase in frictional resistance as the conduits age, corrode, slime, shift and form sediment beds, this range of frictional values was considered validated.

It can be clearly seen that as n increases it has a marked impact on the flow velocities in the pipes. In the most severe case of n = 0.043 (corresponding to a severely slimed pipe) the flow velocity falls to between 0.2 - 0.3 m/s. This is well below the recommended value of 0.65 m/s.

The only plausible conclusion is that as a sewerage network ages it will 'grow' more and more flow impeding features. This will cause greater reductions in velocities and consequently the ability of the sewer to maintain self-cleansing velocities will reduce further and further.

This will in turn increase the ease of sediment settling which in turn will increase friction.

This is the kind of complex causal negative feedback loop which will progressively cause any sewer conduit to tend towards blockage. Justifying why blockages are not attributable to any one cause but rather a critical accumulation of some or all causes listed in the blockage report section.

Considering that the high variability of parameters and interrelated nesting of all of these calculated values, it is clear that there exists massive uncertainty in the calculation procedure of the mechanics of sedimentation.

Without suitable sediment and flow characteristic data collected from localised points in a sewer system in accordance with aging of the sewer system, it is not conceivable that the minimum flow velocity standard of 0.65 m/s can be considered valid for all sewer sections.

As has been shown, Cape Town seems to have a very serious problem with sediment intrusion into its drainage network. This comes with increased blockage numbers and a very expensive blockage clearance bill. It would be of great benefit to the city to better understand the vivirithmic nature of problematic sections of sewer with regard to sediment loading. Once these sections are better understood they may be specifically designed for so as to prevent the problems before they manifest into a costly maintenance fix.

Chapter Six

Conclusion and Recommendations

6.1 Conclusion

While there have been many subsystems displayed and analysed in this research, the overarching focal question was, what is the state of sanitation in South Africa? In addressing this question, it was necessary to provide a general understanding of the history and necessity of functional sanitation in society.

To this end, the definition and conceptualisation of the boundary conditions of a case study city were provided. Followed by an analysis of the macro-patterns of sanitation system decay in the case study, and an evaluation of what is possible in terms of mathematical modelling and understanding, of the effects of one of the macro-patterns, namely sedimentation.

From this, it was deduced that the answer to the problem does not lie solely in the application of hard engineering. When viewing this problem, with an understanding of the complexity of South African society, it was necessary to step beyond the bounds of what is considered 'normal' civil engineering research. The reason being that the systems of decay, which are presented within the body of this research, are a combination of natural and anthropogenic phenomena. It was shown that there is a strong theoretical understanding of the nature of sedimentation and reasonable rules, which are implemented in sewer design to compensate for sedimentation. However, it was further concluded, due to a lack of knowledge of what the true local conditions of sedimentary patterns are, there cannot be a truly effective solution applied to this problem. Instead, a heuristic approach is adopted of assigning a minimum self-cleansing velocity to sewer design. However, through the presented complexity of the nature of sedimentation mechanics, it can be clearly understood that, this heuristic approach will not always provide a suitable solution to sedimentation.

In summary, the findings of this research were this: Cape Town is the most functional sanitation system in the country. It is; however, beleaguered by numerous problems including, an inability for the currently active wastewater treatment plants to process the volumes generated by the rapidly increasing population of the city. This has a concomitant effect of contaminating the proximal hydrosphere of the city. While there has been a case made that the effects of this inability to safely process wastewater are not as severe as is reported, it was shown that there are levels of pathogen and chemical counts, significantly greater than the limits stipulated by national guidelines. This unequivocally points to the validity of the conclusion that, the city's wastewater fleet is incapable of performing to a suitable level of wastewater detoxification. Whether this is directly due to capacity violation or whether it is an inability to functionally collect all wastewater generated in the city is still uncertain. However, it is very likely that it is a combination of both of these problems.

Furthermore, it was presented that, the plight of Eskom, the country's most significant power producer, is in a state so dire and of such a large economical scale that it financially threatens the continued functionality of numerous other systems. Furthermore, from the perspective of power supply, the regularity with which Eskom implements load shedding poses a direct threat to the functionality of the wastewater treatment capabilities of the city. This is due to the city's heavy reliance on pump stations, to re-elevate wastewater from sump points at the end of the gravity network into the wastewater treatment plants. While there are design guidelines, which should ensure that there are suitable power supply redundancies, these redundancies are known to incur significant additional costs on an already financially burdened system.

From a 10 year long dataset of areal blockage patterns, it was shown that there has been a significant trend of blockage event increase. While a significant portion of this blockage event increase may be attributed to the steady growth of the population, it was acknowledged that a further cause of these blockage events is due to an aging system, with an inadequate understanding of the true nature of the blockages. With 99% of the blockage events being described as “Sewer: Blocked/Overflow”, there is a very limited understanding of what is physically causing these blockages. It was concluded that it is likely to be a combination of numerous different blockage causes, including biological sliming, foreign objects becoming trapped and sedimentation reducing the cross sectional area of flow.

Of these listed causes of blockage, the cause with the most significant theoretical understanding is that of sedimentation. Therefore, an analysis of the theoretical mechanics of sedimentation was presented. Due to acknowledgement of decidedly limited data availability and quality it was decided that the presentation of the theory of sedimentation, should be conducted through an uncertainty analysis. To this end, the general ranges of significant variables were extracted from literature or assumed. These ranges were then cycled through the numerous dimensions for equations of sedimentation.

Furthermore, it was shown that an equation for determination of the fall velocity of a sediment particle was many orders of magnitude, outside of what could conceivably be considered reasonable. Despite the literary source of this equation being considered to be of a high standard, this equation was discarded in favour of a more commonly employed, fall velocity equation from a different literary resource.

Through this uncertainty analysis' presentation of the interrelation of many different variables, it was concluded that, the generally accepted notion of a minimum self-cleansing velocity causing over design of pipes, with a diameter of less than 500 mm and the under design of pipes, with a diameter greater than 500 mm.

Furthermore, through this analysis of the case study of Cape Town, as the most efficacious sanitation system in South Africa, it can be inferred that any problems experienced in Cape Town would likely be varying orders of magnitude worse in every other settlement in the country. In the absence of generally sound municipal auditing practices in the majority of South Africa there is a serious lack of knowledge of the trends of blockage creation, clearance and expense for the majority of South African communities. However, with recollection of the R 270 million spent on blockage clearance in Cape Town in 2019 and the knowledge that Cape Town is the most efficacious sanitation system in the country, one can only begin to imagine what financial damage is being wrought on the efficaciousness of the South African sanitation system through intrusion of sediment and trash into what should be a closed, waterborne system. As the system ages these intrusionary events will become more temporally and spatially probable.

Returning to the focal question of this research, what is the state of sanitation in South Africa? This is a truly vast question to answer.

The evidence collected and presented in this research was largely specific to Cape Town. This in large part was due to City of Cape Town readily providing some of the information sought. There is an enormous problem in South Africa of inaccurate or no municipal records being kept. So while some benchmark data was sought for local conditions, of the 9 provincial capitals, only Johannesburg and Cape Town had readily available data.

Without further links into the municipal halls of power of other cities this research was limited to assessing the state of Cape Town's sanitation system. Due to Cape Town being the highest ranking Green Drop community, it was decided logical that any problems prevalent in Cape Town would be prevalent to some degree in other communities.

The major problems identified in Cape Town were; high pollutant counts in the proximal hydrosphere. Capacity violations of the wastewater treatment plants. A heavy reliance on pumping. And very high blockage numbers.

The high pollutant counts were reported in literature as being a combination of watercourse dumping, fouled surface runoff and the possibility of the wastewater treatment plants underperforming.

The analysis of historically processed flows in the wastewater treatment plants gave credence to the argument that the wastewater treatment plants are regularly overloaded and as such some quantity of wastewater is inadequately treated before release.

The reliance of pumping was of concern due to the fragility of power generation by Eskom in the country. In spite of having made a request or information pertaining to back up power at the wastewater treatment plants and the pump stations, the City did not reply with any information.

The blockage numbers presented in this research show that Cape Town has twice the blockage numbers of Johannesburg (a city with a roughly equal population) and some 40 times greater than the international average rate of sewer blockages.

The rate of sewer blockages has been increasing by roughly 4.9 % per year in Cape Town. This shows a strong trend of increase. Therefore from the evidence which was available to this research there seems to be a pronounced trend of decay across numerous different facets of the Cape Town sanitation system. What this means nationally is that all other systems which were objectively graded as less efficacious than Cape Town will be experiencing similar problems likely to a higher degree.

There is some unknown volume of wastewater being generated by South African society. There is a slightly better but still largely unquantified amount of this wastewater which is adequately treated. The rest of the generated wastewater is finding its way into the fresh water resources of the country.

This inability of the South Africa to correctly identify and contain sources of its generated wastewater will become more and more pronounced as the country continues to under-invest in maintenance and upgrades of a most essential infrastructural asset.

What sanitation serves to do for a society is protect the future society from the waterborne waste of today. The knock on effects of inadequate treatment of wastewater are extremely costly in all senses of the word.

What has been shown in this research is the natural degradation and decay of a system which is undervalued by a society. Just as universally potholed roads show a decline in health of the road network of South Africa the marked sanitation problems in Cape Town and reported, yet unconfirmed sanitation problems in other communities imply that the sanitation capacity of the country is in decline.

When considering the increasing fragility of South African society these are truly ominous warning signs. They warn of a entelechial fate similar to that of numerous other African states and other 3rd world countries who undervalued their sanitation and other key infrastructural systems.

Sanitation in South Africa is but one organ of state which enables the society to function in a healthy fashion. In the absence of a functional sanitation system more and more unforeseen water related problems will manifest. Further threatening an already water sensitive country.

Unequivocally these are locally specific signs of the natural decay of societies since the dawn of time. The subtle appearance of the Immutable Law of Societal Termination in South Africa.

Like the spoor of an animal in soft sand, one is only left with a tenuous understanding of what it is that is stalking out there. However, with the application of reason and investigation it is possible to infer that sanitation in South Africa is increasingly threatened. This will be just one more piece of evidence towards the Immutable Law of Societal Termination.

Without a concerted effort to firstly acknowledge these issues an secondly to proactively tackle them the end point of South African sanitation is a foregone conclusion. A rather unpleasant conclusion all things considered.

6.2 Recommendations

6.2.1 Data Collection

In order to create an effective and functional model it is essential that one has valid data. A strong argument was put forward in this research that South Africa is plagued by a decay in quality data collection. In order to design solutions that will work correctly in the reality of South Africa, it is imperative that the data used for the design is of a high standard.

Within the ambit of this research, it is recommended that data for South African sewerage networks be collected with regard to accurately describing the causes of blockages. This would include but not be limited to:

Sediment characteristics (such as median particle sizes, median particle densities and cohesiveness) found in specific networks or problematic sections of networks. This will allow for the calibration and accurate implementation of the sediment equations provided in this research.

Non-sedimentary blockage cause analysis such as fat build up and solid matter lodging.

With this kind of information more specific solutions could be designed for in future networks. Furthermore in existing networks it would be possible to identify sources of blockage material and to minimise and mitigate these sources and their effects.

A more in depth analysis of the efficacy of various key city's wastewater treatment plants. If the wastewater treatment plants are truly as capacity limited as it appears in this research, then there can be no doubt that the mistreated wastewater will find its way into crucial clean water resources. This will have a severe negative impact on the general health of the country going forward.

6.2.2 Deploying Artificial Intelligence and Machine Learning Algorithms in Sanitation Systems

A considerable amount of background research was done into the fields of AI and ML with respect to their applications in sanitation systems. This is a relatively new field of investigation. However, AI and ML techniques have been shown to be extremely powerful in modelling black-box systems and dealing with uncertainties. If done correctly, it is posited that the application of AI and ML techniques could allow for very strong alternative approaches for understanding of such things as blockage patterns and pollutant loads in networks. Again, an undertaking such as this would be inherently limited by the quality of data collected and used for model creation.

6.2.3 Alternative Sanitation System Design

In the Literature Review mention was made of Simple Sewer Design. This is an example of an alternative sewer system design strategy which was tailor-made for the societal constraints of Brazil, a country very similar in social makeup to South Africa. Considering the evidence provided in this research of the scale of corruption inherently found in South Africa and the relatively low national budget apportioned to sanitation system upgrade and maintenance, it would appear that as a country there needs to be a shift in focus to more affordable and efficacious sewer systems than conventional sewer design.

Chapter Seven

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APPENDIX

Appendix A

Selected Wastewater Treatment Plant Average Daily Flowrates

This appendix shows the Average Daily Flowrate graphics for the top 8 biggest wastewater treatment plants and the top 3 biggest sea outfalls for the City of Cape Town.

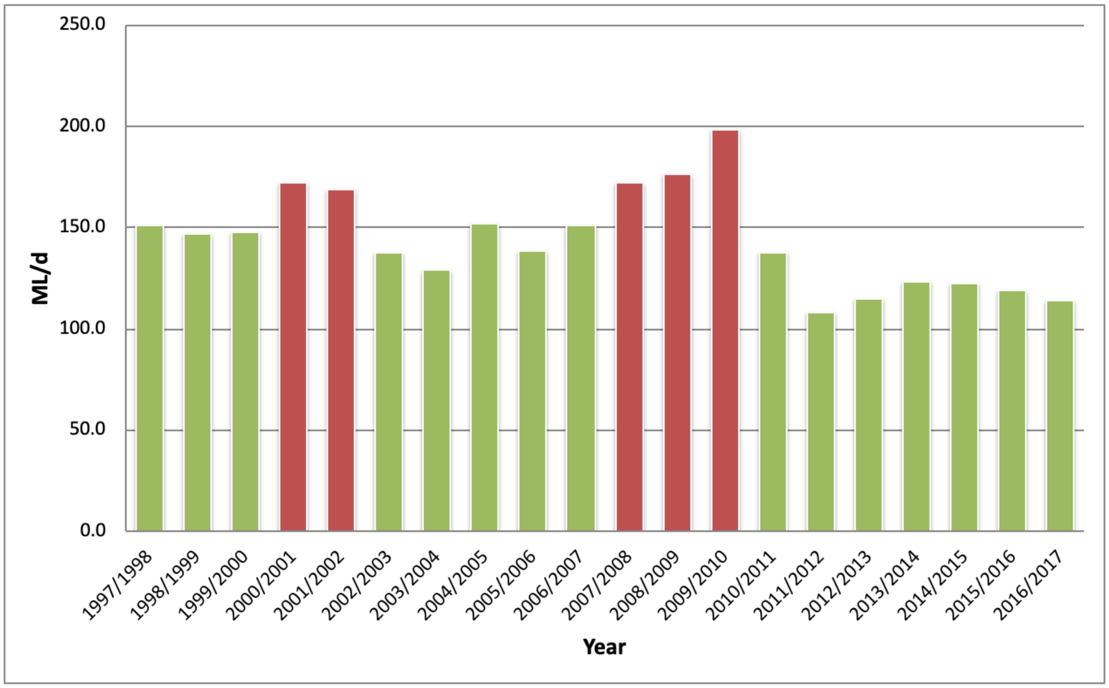

Figure A.1 Cape Flats Wastewater Treatment Plant Flowrates Showing Violated Years

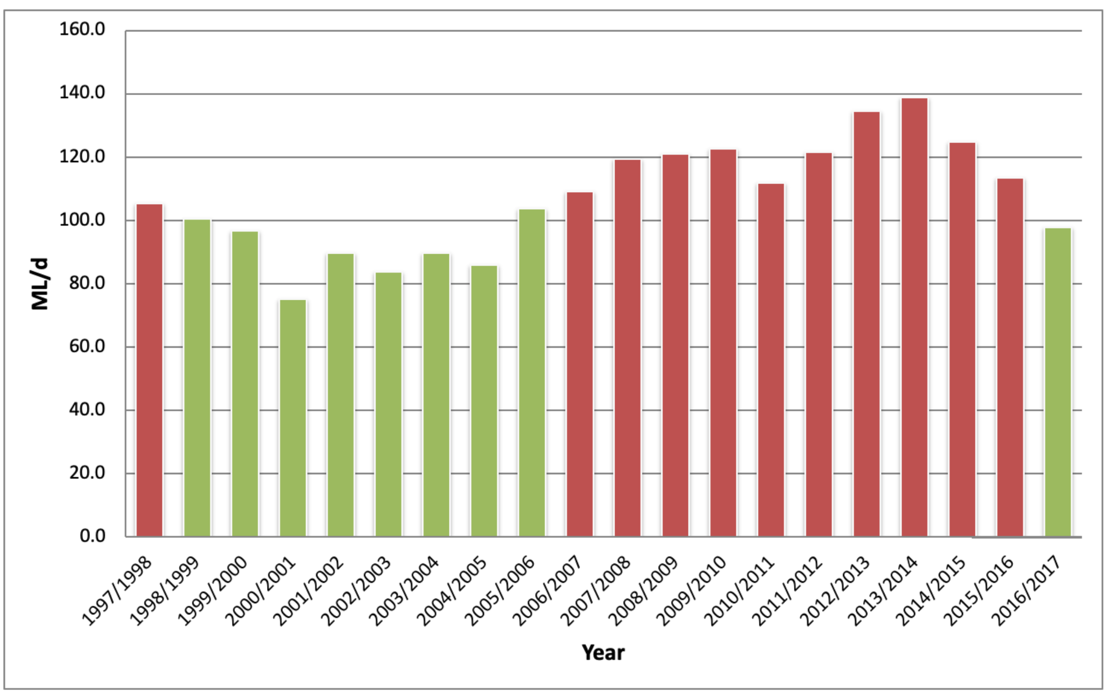

Figure A.2 Athlone Wastewater Treatment Plant Flowrates Showing Violated Years

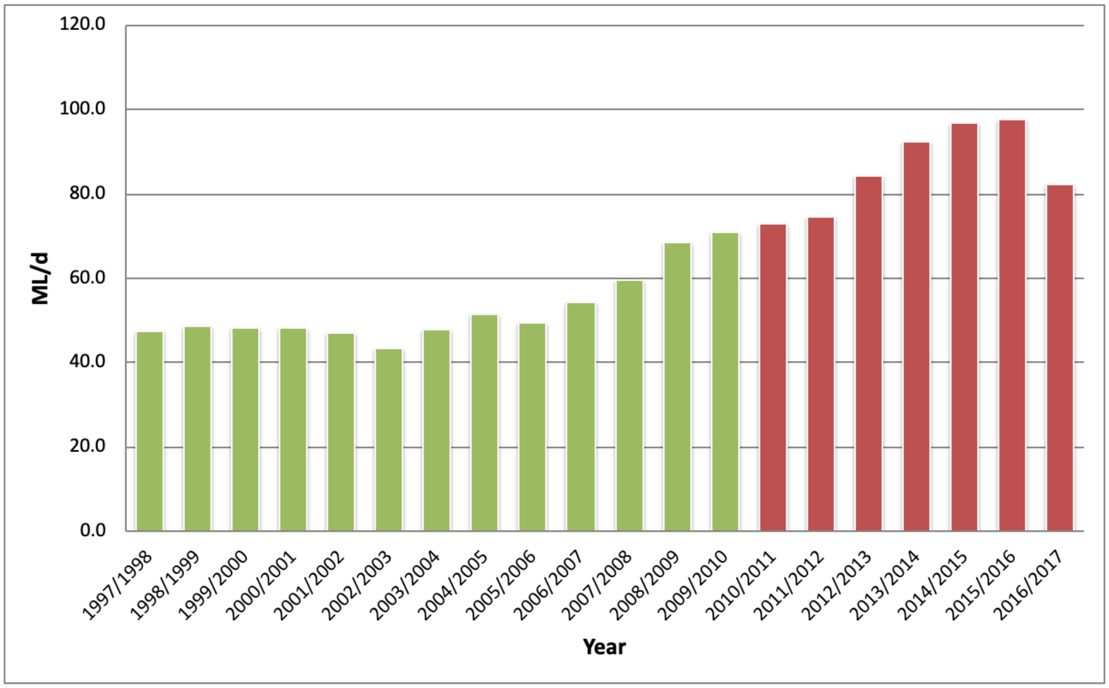

Figure A.3 Zandvliet Wastewater Treatment Plant Flowrates Showing Violated Years

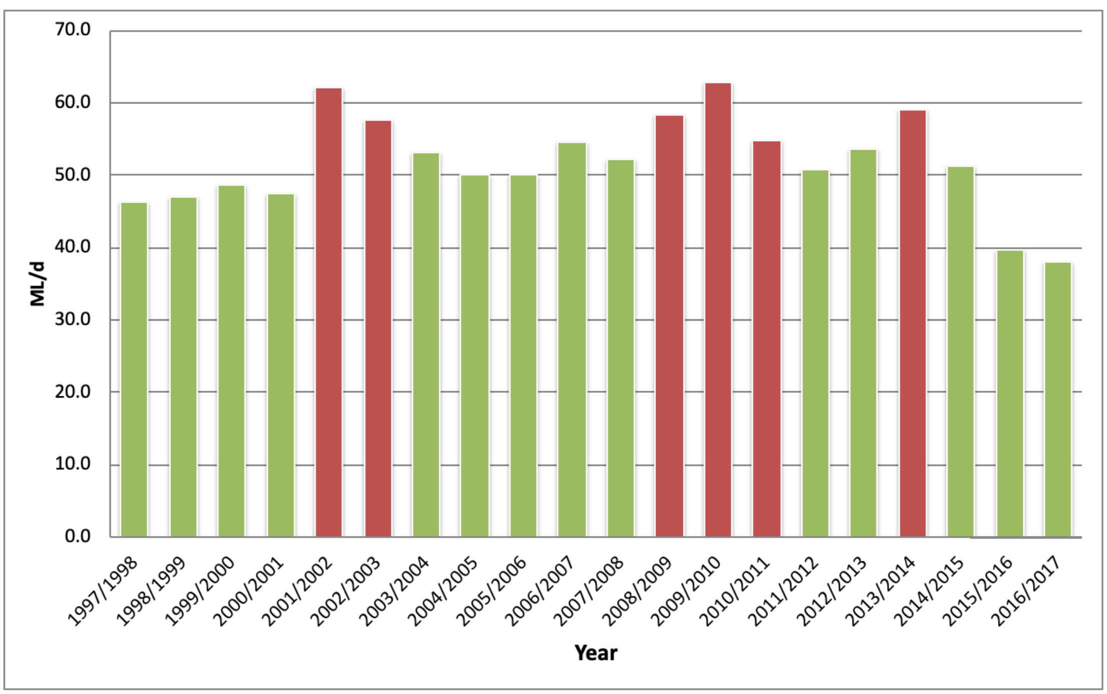

Figure A.4 Bellville Wastewater Treatment Plant Flowrates Showing Violated Years

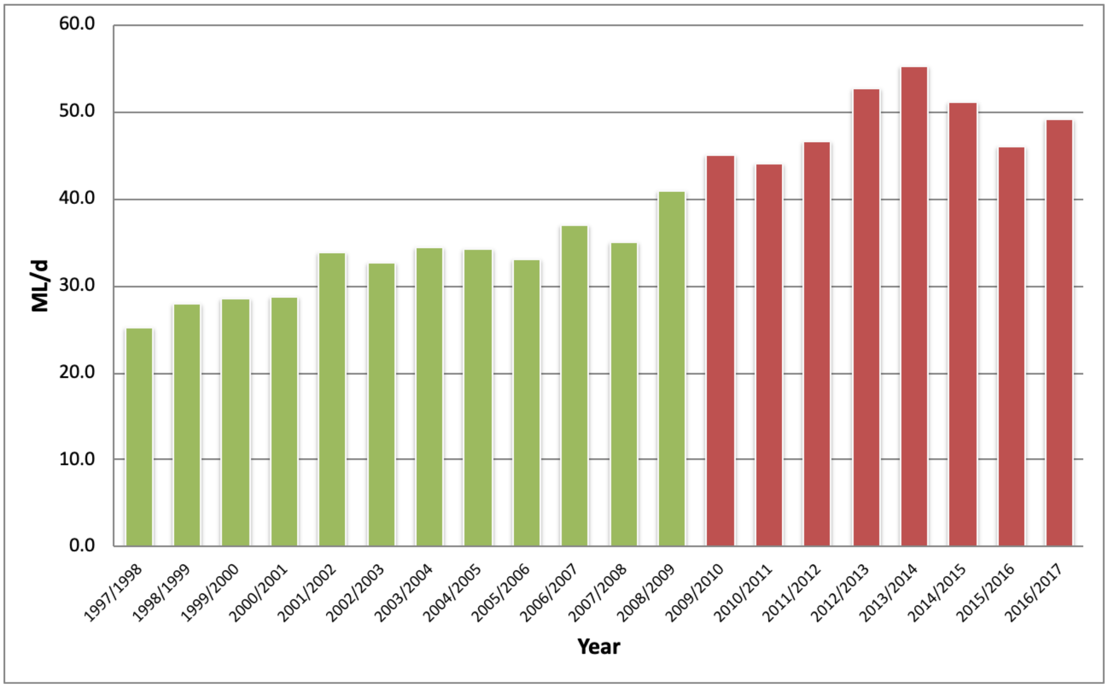

Figure A.5 Potsdam Wastewater Treatment Plant Flowrates Showing Violated Years

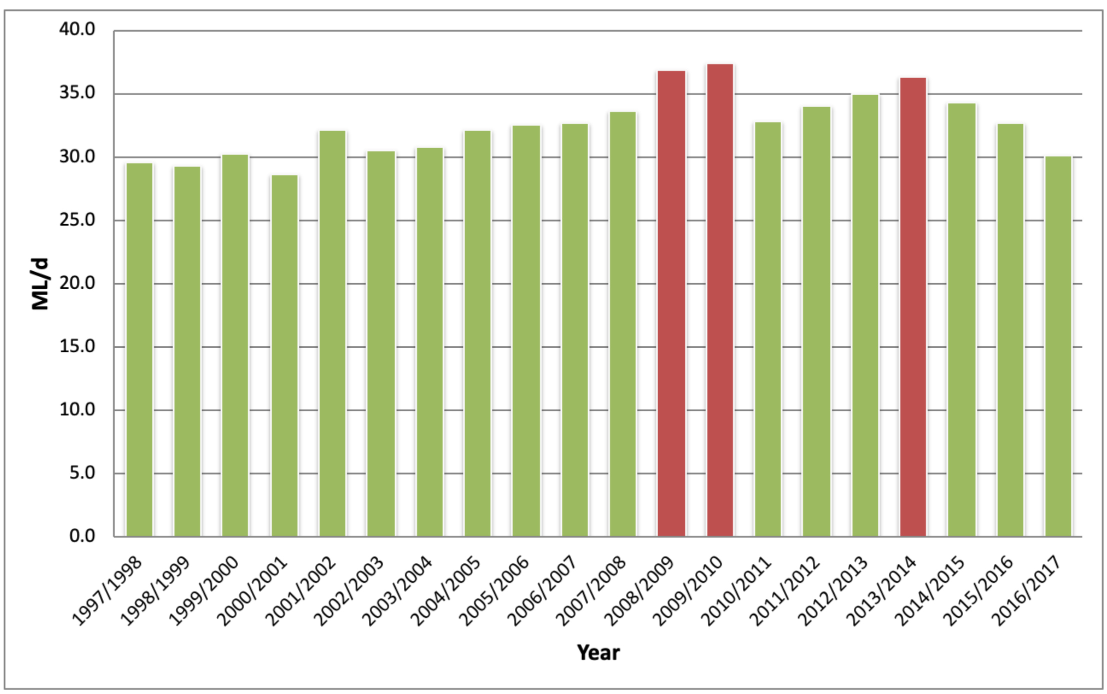

Figure A.6 Mitchell's Plain Wastewater Treatment Plant Flowrates Showing Violated Years

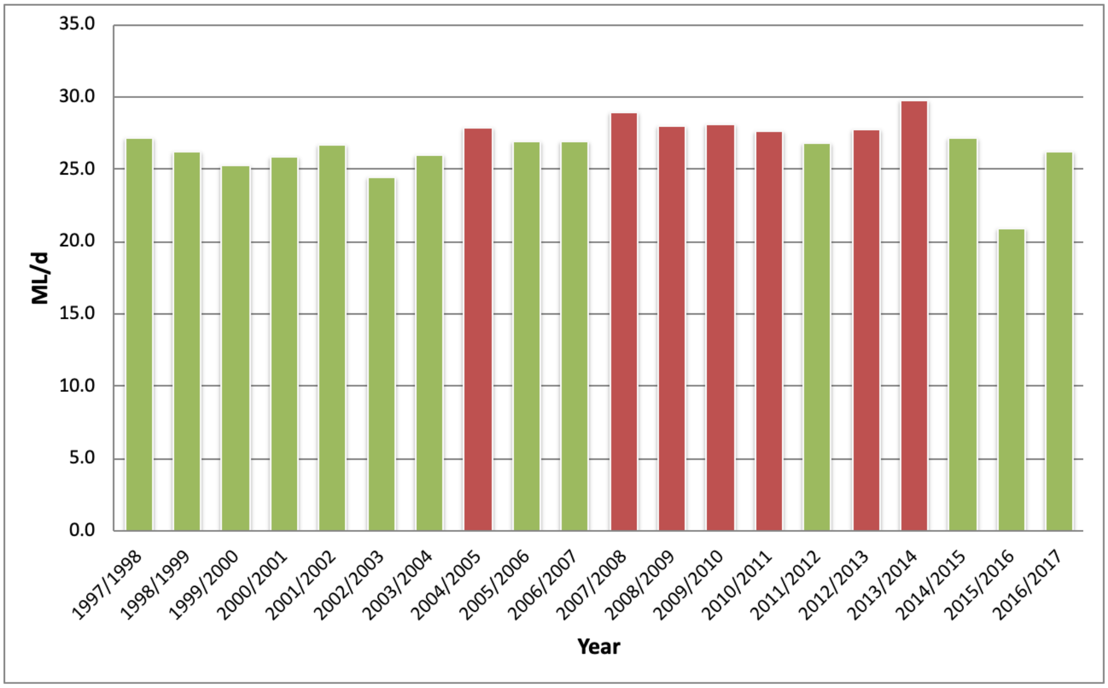

Figure A.7 Greenpoint Outfall Flowrates Showing Violated Years

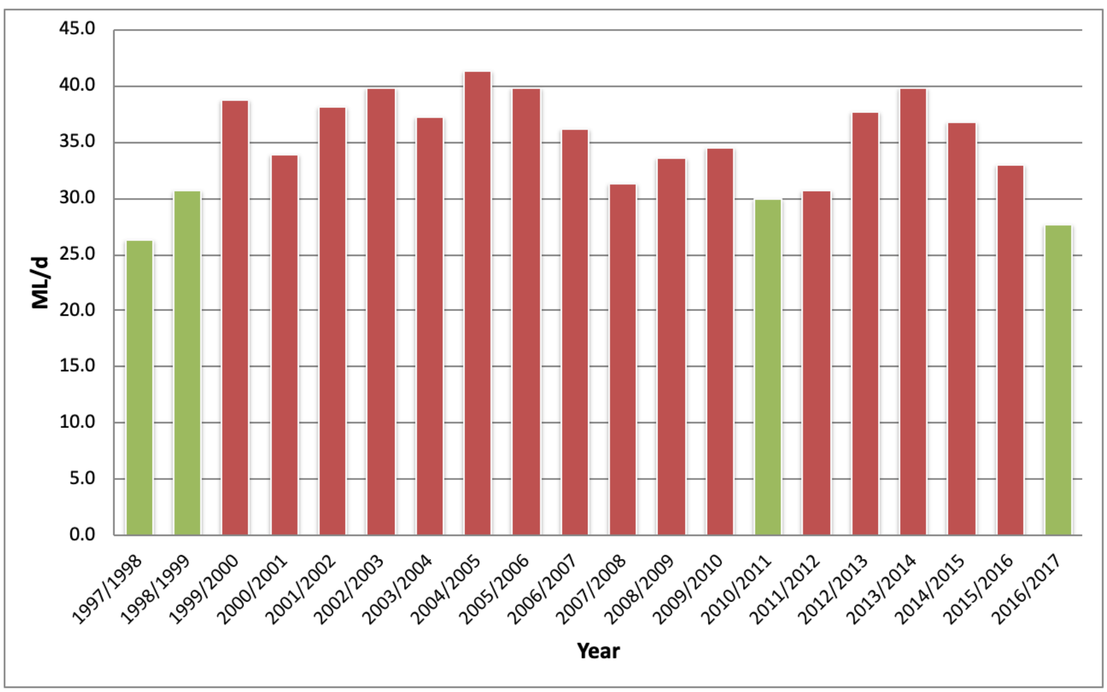

Figure A.8 Macassar Wastewater Treatment Plant Flowrates Showing Violated Years

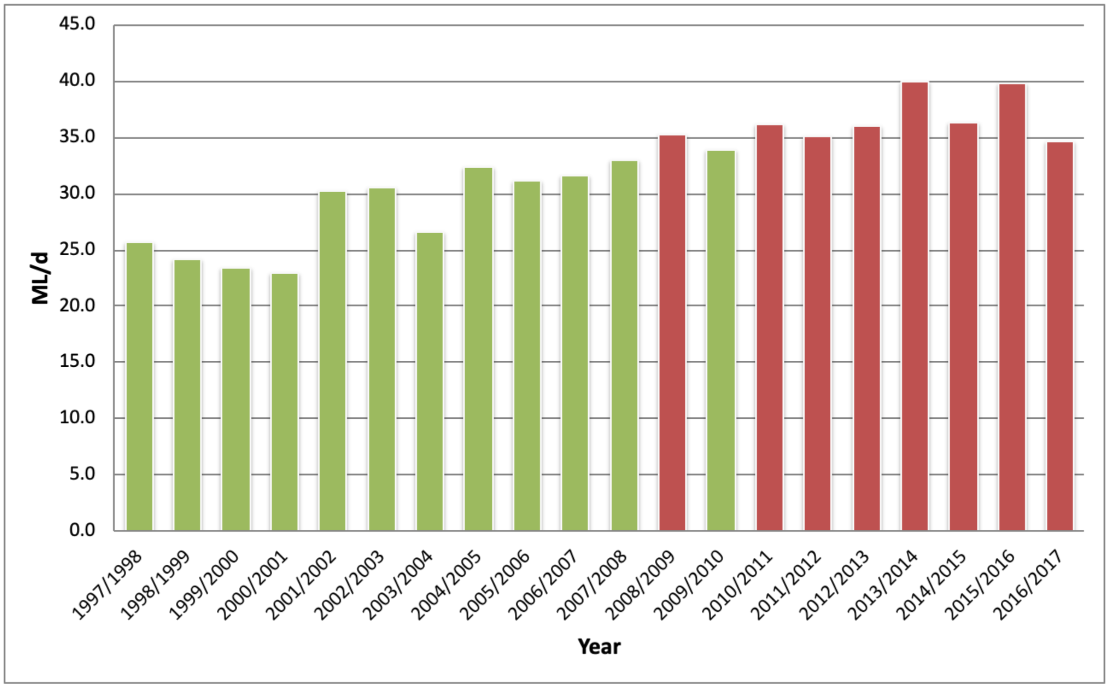

Figure A.9 Borcherd's Quarry Wastewater Treatment Plant Flowrates Showing Violated Years

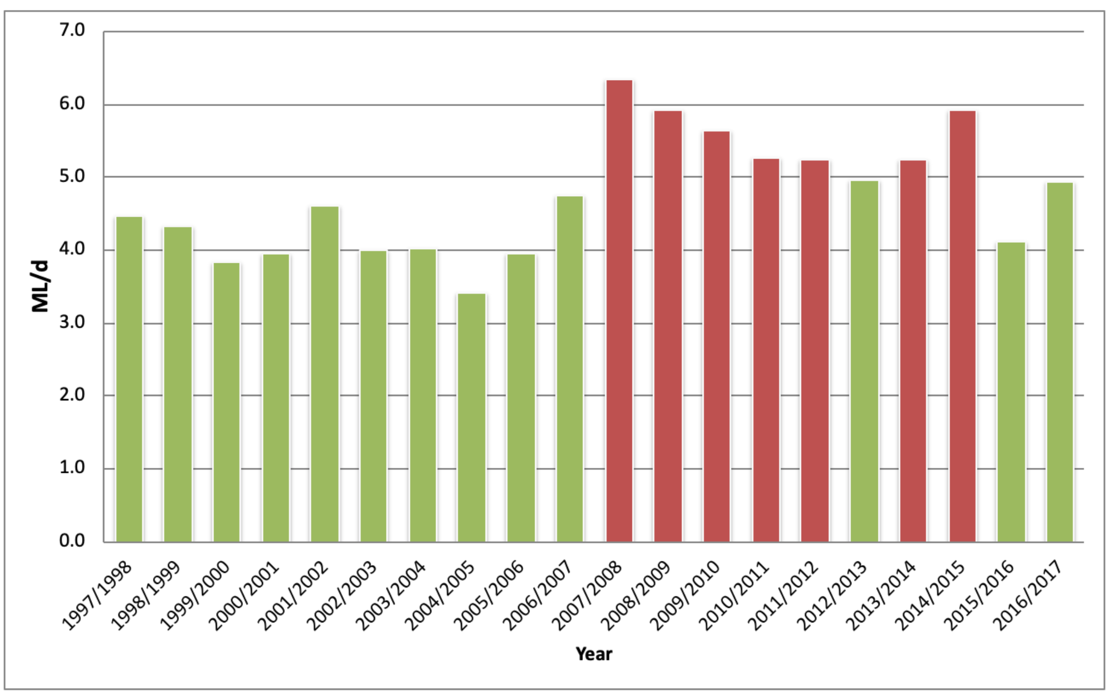

Figure A.10 Hout Bay Outfall Flowrates Showing Violated Years

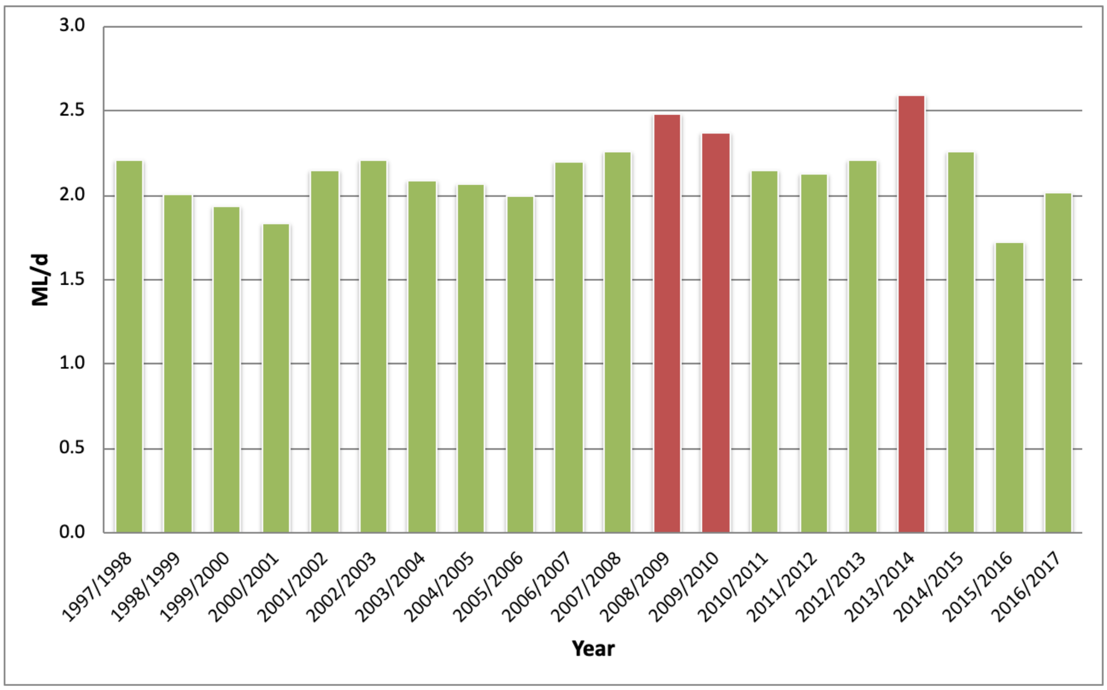

Figure A.11 Camp's Bay Outfall Flowrates Showing Violated Years

Appendix B

Java Code for Mechanics of Sedimentation Equations

This appendix shows the Java Code used to create the solution surfaces of the sedimentation equations.

package sediment$_{T}ransport_{E}quations$;

import java.lang.Math;

public class MSMS {

static double g = 9.81;

//tau$_0 = Boundaryshearstress[N/m2]$

//f = Darcy-Weisbach friction coefficient [Dimensionless]

//rho = fluid density [kg/m3]

//V = mean velocity [m/s]

/*BOUNDARY SHEAR AND FRICTION COEFFICIENT*/

static double getTau$_0(doublef, doublerho, doubleV)\{$

double tau$_0 = f * rho * V * V;$

return tau$_0; \}$

//f = Darcy-Weisbach friction factor [Dimensionless]

//k = boundary roughness [m]

//D = pipe diameter [m]

//v = fluid kinematic viscosity [m2/s]

//g = gravitational constant [m/s2]

//S = slope of hydraulic grade line [Dimensionless]

static double get$_f(doublek, doubleD, doublev, doubleS)\{$

double n = Math.pow(k, 0.16161616)/26;

double f = g*n*n/Math.pow((0.7*D), 0.3333);

// double f = Math.pow((1/(-2*Math.log(k/(3.7*D)+(2.51*v)/(D*Math.pow((2*g*D*S), 0.5))))), 2);

return f; }

//kb = Bed roughness [m]

//D = pipe diameter [m]

//d$_5$0 = $medianparticlesize[m]$

static double get$_k b(doubleD, doubled_5 0)\{$

double kb = 2.4*Math.pow(D, 0.61)*Math.pow(d$_5 0, 0.39$);

return kb; }

//fc = Composite friction factor [Dimensionless]

//f = Darcy-Weisbach friction factor [Dimensionless]

//P = wetted perimeter [m]

//fb = bed friction factor (computed with kb) [Dimensionless]

//Wb = sediment bed width [m]

static double get$_f c(doublef, doubleP, doublefb, doubleWb)${

double fc = (f*P+fb*Wb)/(P+Wb);

return fc; }

/*NON-COHESIVE SEDIMENT MOVEMENT*/

/*SUSPENDED SEDIMENTS*/

//Rouse-Vanoni suspended load vertical distribution

//Ca, Cd = Concentration at distances a and d from the bed, respectively [V/V]

//D = stream depth [m]

//y = distance from stream bed [m]

//d = distance from stream bed [m]

//z = suspension number [Dimensionless]

//if z -> 0 vertical suspension distribution is uniform//

//if z -> 5 the suspended load is insignificant and bedload is primary form of transport

double getCy(double Ca, double D, double y, double a, double z){

double Cy = Ca*Math.pow((((D-y)/y)*(a/(D-a))), z);

return Cy; }

//z = Suspension number

//w = particle fall velocity [m/s]

//k = von Karman constant (usually 0.4 for clear water)

//Ustar = shear velocity [m/s]

static double get$_z(doublew, doublekarman, doubleUstar)${

double z = w/(karman*Ustar);

return z; }

/*SHEAR VELOCITY*/

//tau$_0 = boundaryshearstress[N/m2]$

//rho = fluid density [kg/m3]

static double getUStar(double tau$_0, doublerho)\{$

double UStar = Math.pow((tau$_0/rho), 0.5)$;

return UStar; }

/*FALL VELOCITY*/

//w = particle fall velocity [m/s]

//v = fluid kinematic viscosity [m2/s]

//d$_5$0 $= medianparticlesize[mm]$

//s = specific gravity [Dimensionless]

static double get$_w(doublev, doubled_50, doubles)\{$

double w = Math.pow(((9*Math.pow(v, 2)+Math.pow(10, 9)*Math.pow(d$_5 0, 2) * g * (s - 1) * (0.03869 + 0.0248 * d_5 0) - 3 * v)), 0.5)/(Math.pow(10, -3) * (0.11607 + 0.074405 * d_5 0))$;

return w; }

static double get$_w 2(doublev, doubled_5 0, doubles)\{$

double D$_s tar = 0$;

D$_s tar = Math.pow(((g * (s - 1))/Math.pow(v, 2)), 0.3333) * d_5 0$;

double w = v/d$_5 0 * (Math.sqrt(Math.pow(10.36, 2) + 1.049 * Math.pow(D_s tar, 3)) - 10.36)$;

return w; }

/*LIMIT OF DEPOSITION*/

//Macke formula for low concentrations at limit of deposition

//Cv = volumetric concentration (volumetric discharge of sediment/volumetric discharge of fluid)

//fc = composite friction factor [Dimensionless]

//Vl = limiting velocity without deposition [m/s]

//s = specific gravity [Dimensionless]

//w = particle fall velocity [m/s]

//A = cross sectional area of flow (m2)

static double getMackeCv(double fc, double Vl, double s, double w, double A){

double Cv = Math.pow(fc, 3)*Math.pow(Vl, 3)/(30.4*(s-1)*Math.pow(w, 1.5)*A);

return Cv; }

/*DEPOSITED BED*/

//Ackers-White formula for flow with some deposition, low concentrations

//Cv = volumetric concentration (volumetric discharge of sediment/volumetric discharge of fluid)

//Wb = bed width [m]

//We = effective width of sediment bed [m]

//R = hydraulic radius [m]

//A = cross sectional area of flow [m2]

//$d_50 = medianparticlesize[mm]$

//fc = composite friction factor [Dimensionless]

//V = mean flow velocity [m/s]

//s = specific gravity [Dimensionless]

//D = internal pipe diameter [m]

//proportionSed = proportion of depth of flow as sediment bed

//v = fluid kinematic viscosity [m2/s]

//ys = mean sediment depth over pipe invert [m]

//Dgr = dimensionless grain size

//Agr, H, J, K, m, alpha, beta, gamma, delta, epsilon dimensionless factors based on Dgr

//n = Manning's roughness coefficient

static double getAckersWhiteCv(double Wb, double R, double A, double d$_5$0, $doublefc, doubleV, doubles, do$

double ys = proportionSed*D;

double We, m, n, Agr, H;

if((0.01 <= proportionSed) (proportionSed <= 0.1)){ We = (0.2+3.33*(ys/D-0.01))*Wb;
}else{ We = 0.5*Wb;

}

double Dgr = Math.pow((g*(s-1)/Math.pow(v, 2)), 1/3)*d$_5$0;

if(Dgr<=60){ m = 1.67+0.83/Dgr;

H = Math.pow(Math.E, (2.79*Math.log(Dgr)-0.98*Math.pow(Math.log(Dgr), 2)-3.46));

n = 0.012;

Agr = 0.14+0.23/Math.pow(Dgr, 0.5); }else{ m = 1.78;

n = 0;

Agr = 0.17;

H = 0.025; }

double J = (Math.pow(8, (n*(1-m)/2)*H))/(Math.pow(113, (m*(1-n))*Math.pow(Agr, m)));

double alpha = 1-n;

double beta = (10-4*m-m*n)/10;

double gamma = n*(m-1)/2;

double K = Math.pow(11.3, (1-n))*Math.pow(g, n/2)*Agr;

double delta = -n/2;

double epsilon = (4+n)/10;

double Cv = J*Math.pow((We*R/A), alpha)*Math.pow((d$_5 0/R), beta)*Math.pow(fc, gamma)*$
$Math.pow((V/Math.pow((g*(s-1)*R), 0.5))-K*Math.pow(fc, delta)*Math.pow(d_5 0/R, epsilon), m);$

return Cv; }

//Shields parameter. Criterion for threshold of movement

//tau$_0 = Boundaryshearstress[N/m2]$

//tauStar = Shields parameter [Dimensionless]

//rho$_s = sedimentdensity[kg/m3]$

//rho = fluid density [kg/m3]

//d$_5 0 = medianparticelsize[mm]$

static double getTauStar(double tau$_0, doublerho_s, doublerho, doubled_5 0)\{$

double tauStar = tau$_0/(g*(rho_s - rho)*d_5 0);$

return tauStar; }

/*LIMIT FOR DEPOSITION*/

//May concentration formula for limit of deposition (no bed)

//Cv = volumetric concentration (volumetric discharge of sediment/volumetric discharge of fluid)

//D = internal diameter [m]

//A = cross sectional flow area [m2]

//d$_5 0 = medianparticlesize[mm]$

//V = mean flow velocity [m/s]

//Vt = mean flow velocity at threshold of movement [m/s]

//s = specific density [Dimensionless]

//$y_0 = flowdepth[m]$

static double getMayLimCv(double D, double A, double $d_50, doubleV, doubles, doubley_0$){

double Vt = 0.125*Math.pow((g*(s-1)*$d_50), 0.5) * Math.pow(y_0/d_50, 0.47)$;

double Cv = 3.03*Math.pow(10, -2)*(Math.pow(D, 2)/A)*Math.pow($d_50/D, 0.6)*Math.pow((1-Vt/V)/V, 4)*Math.pow(Math.pow(V, 2)/(g*(s-1)*D), 1.5)$;

return Cv; }

/*DEPOSITED BED*/

//May concentration formula for deposited bed

//Cv = volumetric concentration (volumetric discharge of sediment/volumetric discharge of fluid)

//Wb = sediment bed width [m]

//D = internal pipe diameter [m]

//A = cross sectional area of flow [m2]

//nu = transport parameter for continuous bed

//fg = Darcy-Weisbach friction factor for grain shear stress

//theta = transition coefficient for particle Reynolds number

//Fg = grain mobility factor

//$R_starc = particleReynoldsnumber$

//Fs = effective sediment mobility

//V = mean flow velocity

//s = specific gravity

//d$_5$0 $= medianparticlesize$

//v = fluid kinematic viscosity

static double getMayDepCv(double Wb, double D, double A, double V, double s, double d$_5$0$, doublev, doubleS)${

double nu, theta, fg, Fg, Fs, R$_s tarc$;

fg = get$_f((1.23 * d_5 0), D, v, S)$;

Fg = Math.pow((fg*Math.pow(V, 2))/(8*g*(s-1)*d$_5$0), 0.5);

R$_s tarc = Math.pow(fg/8, 0.5) * (V * d_5 0/v)$;

theta = (Math.exp(R$_s tarc/12.5) - 1)/(Math.exp(R_s tarc/12.5) + 1)$;

Fs = Fg*Math.pow(theta, 0.5);

if(Fs <= 0.1){

nu = 0;

}else if((0.1 < Fs) (Fs <= 0.225)){

nu = 1.2*(Fs-0.1);

}else if((0.225 < Fs) (Fs <= 0.275)){

nu = 0.15+9*(Fs-0.225);

}else if((0.275 < Fs) (Fs <= 0.4)){

nu = 0.6+3.2*(Fs-0.275);

}else if((0.4 < Fs) (Fs <= 0.7)){

nu = 1-(Fs-0.4);

}else{

nu = 0.7; }

double Cv = nu*(Wb/D)*(Math.pow(D, 2)/A)*((theta*fg*Math.pow(V, 2))/(8*g*(s-1)*D));

return Cv; }

/*NEAR BED SOLIDS FLUSH*/

//Ashley near bed solids concentration combined sewer

//Cv = volumetric concentration (volumetric discharge of sediment/volumetric discharge of fluid)

//It = rainfall intensity [mm/h]

//TSSS = time since start of last storm [h]

//Dt = total depth of rainfall [mm]

//$\mathrm{y}_0 = averageflowdepth[m]$

//$\mathrm{y}_max = maximumflowdepthforanaveragedryweatherflowday[m]$

//$\mathrm{tau}_0 = averageshearstress[N/m2]$

//$\mathrm{tau}_b = bedshearstress[N/m2]$

//$\mathrm{rho}_d = densityofnearbeddrysolids[kg/m3]$

//$\mathrm{rho}_w = sewagedensity[kg/m3]$

static double getAshleyCv(double It, double TSSS, double Dt, double $\mathrm{y}_0, doubley_max, doubletau_0, doubletau$

double Cv = -105.73+2.55*Math.pow(10, -3)*(It*TSSS/Dt)+0.02023*($\mathrm{y}_0/y_max) + 47.808 * (tau_0/tau_b) + 120.45 * (rho_d/rho_w)$;

return Cv; }

/*COHESIVE SEDIMENT MOVEMENT*/

//Erossion rate estimate for consolidated cohesive beds

//M = rate constant [Dimensionless]

//$\mathrm{tau}_b = bedshearstress[N/m2]$

//tau$_s = bedshearstrength[N/m2]$

static double getE(double M, double tau$_b, doubletau_s)\{$

double E = M*(tau$_b - tau_s)/tau_s$;

return E; }

/*INCIPIENT MOTION IN CIRCULAR SEWERS*/

//Modified Shields parameter. Criterion for threshold of movement cohesive sediments

//tau$_0 = Boundaryshearstress[N/m2]$

//tauStarc = Shields parameter [Dimensionless]

//rho$_s = sedimentdensity[kg/m3]$

//rho = fluid density [kg/m3]

//d$_5 0 = medianparticelsize[mm]$

//tau$_s = shearstrengthofmaterial[N/m2]$

static double getCompositeTau$_s tarc(doubletau_0, doubletau_s tarc, doublerho_s, doublerho, doubled_5 0, doubleta$

double compositeTau$_s tarc = tau_s tarc + tau_s/(g * (rho_s - rho) * d_5 0)$;

return compositeTau$_s tarc; \}$

/*DESIGN CRITERIA U.S. STANDARDS*/

static double getVm1(double K, double s, double d$_5 0, doublef)\{$

double Vm = Math.pow((8*K*g*(s-1)*d$_5 0)/f, 0.5)$;

return Vm; }

static double getVm2(double K, double s, double d$_5 0, doubleR, doublen)\{$

double Vm = Math.pow((K*(s-1)*d$_5 0), 0.5) * Math.pow(R, 1/6)/n$;

return Vm; }

static double getSm(double f, double Vm, double R){

double Sm = (f*Math.pow(Vm, 2))/(4*R*2*g);

return Sm; }

static double getTau$_m(doublerho, doubleK, doubles, doubled_50)\{$

double tau$_m = rho * g * K * (s - 1) * d_50;$

return tau$_m$;

}

static double getVfs(double v, double d$_50, doubles)\{$

double d$_50temp = d_50/1000; doubled_star = Math.pow((g*(s-1)/Math.pow(v, 2)), 0.3333)*$ d_50temp;

// System.out.println("d$_star$ = " + d_star);

double Vfs = v/d$_50temp*(Math.pow((Math.pow(10.36, 2)+1.049*Math.pow(d_star, 3)), 0.5)-$ 10.36);

return Vfs; }

public static void main(String[] args) {

double tau$_0$[] = $\{0.2, 0.4, 0.6, 1, 1.4, 1.8, 2.2, 2.6, 3, 3.4, 3.8, 4.2, 4.6, 5, 6, 7, 8, 9, 10\}; //boundaryshearstress$

double rho = 1000; //fluid density

double V[] = {0.3, 0.5, 0.6, 0.65, 0.7, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.5, 3, 3.5, 4, 4.5, 5, 5.5}; //mean velocity

double f[] = {0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0}; //friction factor

double kb[]; //bed roughness

double fc[]; //composite friction factor double k[] = 0.03, 0.06, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1, 1.1, 1.2, 1.3, 1.4, 2, 4, 6; //absolute roughness

double D[] = {0.110, 0.160, 0.200, 0.250, 0.315, 0.355, 0.450, 0.525, 0.600, 0.675, 0.750, 0.825, 0.900, 1.05, 1.2, 1.35, 1.5, 1.65, 1.8}; //internal pipe diameter

double v[] = {1.79*Math.pow(10, -6), 1.31*Math.pow(10, -6), 1.139*Math.pow(10, -6), 0.89*Math.pow(10, -6), 0.8*Math.pow(10, -6), 0.66*Math.pow(10, -6)}; //fluid kinematic viscosity

double S[] = {8.33E-3, 5E-3, 4E-3, 2.86E-3, 2E-3, 1.66E-3, 1.43E-3, 1.25E-3, 9.09E-4, 7.69E-4, 6.66E-4, 5.55E-4, 5E-4, 4.35E-4, 3.57E-4, 2.94E-4, 2.5E-4, 2.17E-4, 1.89E-4}; //slope

double $d_5 0[] = \{0.01, 0.02, 0.03, 0.04, 0.05, 0.06, 0.08, 0.09, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1, 1.2, 1.4, 1.6,$

double P[]; //wetted perimeter

double fb[]; //bedded composite friction factor

double Wb[]; //average width of bed

double Cy[]; //concentration at distance y from invert

double Ca[]; //concentration at distance a from invert

double y[]; //distance from invert

double a[]; //distance from invert

double z[]; //suspension number

double w[] = {0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0}; //particle fall velocity

double karman = 0.4; //von karmon constant

double Ustar[]; //shear velocity

double s[] = {1.01, 1.1, 1.4, 1.6, 2, 2.3, 2.5, 2.6, 2.7}; //specific gravity

double Vl[]; //limiting velocity

double Cv[] = {0.099, 0.071, 0.063, 0.347, 0.250, 0.219, 0.495, 0.357, 0.313, 0.045, 0.025, 0.020, 0.318, 0.175, 0.140, 0.909, 0.500, 0.400, 0.004, 0.004, 0.004, 0.022, 0.019, 0.019, 0.087, 0.077, 0.074}; //Volumetric concentration

double A[]; //cross sectional flow area

double R[] = {0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0}; //hydraulic radius

double proportionSed[] = {0.005, 0.01, 0.015, 0,02}; //proportion of pipe diameter as sediment

double rho$_s$[]; $//sedimentdensity$

double y$_0$[]; $//averageflowdepth$

double It[]; //intensity of storm

double TSSS[]; //time since start of last storm

double Dt[]; //total depth of precipitation

double y$_m ax$[]; $//maximumflowdepth$

double tau$_b$[]; $//bedshearstress$

double rho$_d$[]; $//densityofnearbeddrysolids$

double rho$_w = 1000; //densityofsewage$

double E[] = {0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0}; //erosion rate

double M[] = {Math.pow(10, -4), Math.pow(10, -3), Math.pow(10, -2)}; //rate constant

double tau$_s[] = \{0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10\}; //bedshearstrength$

double compositeTau$_s tarc$[];

double tau$_s tarc$[]; $//shieldsparameterfromshieldscurve$

double K[] = {0.04, 0.08, 0.12, 0.16, 0.2, 0.24, 0.28, 0.32, 0.36, 0.4, 0.44, 0.48, 0.52, 0.56, 0.6, 0.64, 0.68, 0.72, 0.76, 0.8}; //US design factor double n[] = {0.008, 0.01, 0.012, 0.014, 0.016}; //Mannings roughness coefficient

double Vm[] = {0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0}; //self-cleansing minimum velocity

double Sm[] = {0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0}; //self-cleansing minimum slope

double tau$_m$[] = $\{0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0\}; //self-cleansingminimumboundaryshearstress$

/*FRICTION FACTOR VARIATION*/

// System.out.print("k, v, D, S, f");

// System.out.println();

//

// for(int h = 0; h<k.length; h++){

//

// for(int i = 0; i < D.length; i++){

// f[i] = get$_f(k[h], D[i], v[2], S[i]);$

// if(f[i] <= 1){

// System.out.print(k[h] +", " +v[2] +", " +D[i] +", " +S[i] +", " +f[i]);

// }

// else{

// System.out.print(k[h] +", " +v[2] +", " +D[i] +", " +S[i] +", 1");

// }

// System.out.println();

// }

// System.out.println();

// }

// for(int i = 0; i<v.length; i++){

// f[i] = get$_f(k[i], D[18], v[i], S[18])$;

// System.out.println(f[i]);

// }

/*BOUNDARY SHEAR STRESS VARIATION*/

// System.out.println(V.length);

System.out.print("k, f, S, D, V, tau$_0$");

System.out.println();

for(int i = 0; i<4; i++){ for(int j = 0; j<k.length; j++){

f[j] = get$_f(k[j], D[i*5], v[2], S[i*5])$;

for(int l = 0; l<V.length; l++){

tau$_0[l] = getTau_0(f[j], rho, V[l]); System.out.println(k[j] + "," + f[j] + "," + S[i*5] + "," + D[i*5] + "," + V[l] + "," + tau_0[l]); \} System.out.println(); \}\}$

// /*FALL VELOCITY*/

// System.out.println("v, d$_5 0, s, w$");

// System.out.println();

// for(int i = 0; i<v.length; i++){

// for(int j = 0; j<s.length; j++){

// for(int l = 0; l<d$_5 0.length; l++)\{$

//

// w[l] = get$_w(v[i], d_5 0[l]/1000, s[j])$;

```
// System.out.println(v[i] +", " +d_50[l]/1000 + "," + s[j] + "," + w[l]);
// }
// System.out.println();
// }
// }
/*US DESIGN CRITERIA VARIATION*/
//Vm
// s[0] = 1.01;
// s[1] = 2.7;
// d_50[0] = 0.2;
// d_50[1] = 0.6;
//
// f[0] = 0.0478; //D = 0.16, k = 0.06, S = 0.05
// f[1] = 0.0192; //D = 0.6, k = 0.06, S = 9.09E-4
// R[0] = (Math.PI*Math.pow(0.08, 2))/(Math.PI*0.16);
// R[1] = (Math.PI*Math.pow(0.3, 2))/(Math.PI*0.6);
//
// for(int g = 0; g<1; g++){
// // System.out.println("s = " +s[g]);
// // System.out.println("d_50 = " + d_50[g]);
// // System.out.println("f = " +f[g]);
// // System.out.println("K = , Vm1 = ,");
```

```
// for(int h = 0; h<20; h++){
// Vm[h] = getVm1(K[h], s[g], d50[g], f[g]);
// // System.out.print(K[h] +", " +Vm[h]);
// // System.out.println();
// }
// // System.out.println();
// }
// System.out.println();
// for(int g = 0; g<2; g++){
// System.out.println("s = " +s[g]);
// System.out.println("d50 = " + d50[g]);
// System.out.println("f = " +f[g]);
// for(int j = 0; j<5; j++){
// System.out.println("n = " +n[j]);
// System.out.println("K = , Vm2 = ,");
// for(int h = 0; h<20; h++){
// Vm[h] = getVm2(K[h], s[g], d50[g], R[g], n[j]);
// System.out.print(K[h] +", " +Vm[h]);
// System.out.println();
// }
// System.out.println();
// }
```

```
// }
// for(int g = 0; g<2; g++){
// System.out.println("R = " +R[g]);
// System.out.println("f = " +f[g]);
// System.out.println("Vm = , Sm = ,");
// for(int h = 0; h<20;h++){
// Sm[h] = getSm(f[g], Vm[h], R[g]);
// System.out.print(Vm[h] +", " +Sm[h]);
// System.out.println();
// }
// System.out.println();
// }
// for(int g = 0; g<d_50.length;g++){
// System.out.println("d_50 = " + d_50[g]);
// for(int h = 0; h<s.length; h++){
// System.out.println("s = " +s[h]);
// System.out.println("K = , tau_m =,");
// for(int i = 0; i<K.length; i++){
// tau_m[i] = getTau_m(rho, K[i], s[h], d_50[g]);
// System.out.println(K[i] +", " +tau_m[i]);
// }
// System.out.println();
```

```
// }
// }
// s[0] = 1.01;
// s[1] = 2.65;
// for(int g = 0; g<2; g++){
// System.out.println("s = " +s[g]);
// for(int h = 0; h<v.length; h++){
// System.out.println("v = " +v[h]);
// System.out.println("d_50 =, w =,");
// for(int i = 0; i<d_50.length; i++){
// w[i] = getVfs(v[h], d_50[i], 2.65);
// System.out.println(d_50[i] + "," + w[i]);
// }
// System.out.println();
// }
// }
// for(int h = 0; h<M.length; h++){
// System.out.println("M = " +M[h]);
// for(int j = 0; j<tau_s.length; j++){
// System.out.println("tau_s = " + tau_s[j]);
// System.out.println("tau_b =, E =,");
// for(int i = 0; i<tau_0.length; i++){
```

// E[i] = getE(M[h], $\mathrm{tau}_0[i], tau_s[j]$);

// System.out.println($\mathrm{tau}_0[i] +$ ”, ” $+ E[i]$);

// }

// System.out.println();

// }

// System.out.println();

// }

}

}

Appendix C

Java Code for Manning's Equation

This appendix shows the Java code use to create the solution surface of the Manning's equation varying with n, slope and diameter.

package velocity$_{V} ariations$;

import java.lang.Math;

import javax.swing.text.AbstractDocument.LeafElement;

public class VelocityVariations {

public static void main(String[] args) {

double Q [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0};

double V [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0};

double n [] = {0.01, 0.013, 0.015, 0.02, 0.025, 0.03, 0.035, 0.04, 0.043};

double slope = 0.001;

double S [] = {8.33E-3, 5E-3, 4E-3, 2.86E-3, 2E-3, 1.66E-3, 1.43E-3, 1.25E-3, 9.09E-4, 7.69E-4, 6.66E-4, 5.55E-4, 5E-4, 4.35E-4, 3.57E-4, 2.94E-4, 2.5E-4, 2.17E-4, 1.89E-4};

double D [] = {0.110, 0.160, 0.200, 0.250, 0.315, 0.355, 0.450, 0.525, 0.600, 0.675, 0.750, 0.825, 0.900, 1.05, 1.2, 1.35, 1.5, 1.65, 1.8};

double d [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0};

```
for(int i = 0; i < D.length; i++){ d [i] = 0.7*D[i]; }

double theta [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0};

for(int i = 0; i < D.length; i ++){ theta [i] = 2*Math.acos(1 - 2*(d[i]/D[i])); }

double A [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0};

for(int i = 0; i < D.length; i ++){ A [i] = Math.pow(D[i], 2)*((theta [i] - Math.sin(theta
[i]))/8); }

double P [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0}; for(int i = 0; i < D.length;
i++){ P [i] = theta [i]*D [i]; }

double R [] = {0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0}; for(int i = 0; i < D.length;
i++){ R [i] = (D [i]/4)*(1-Math.sin(theta [i])/theta [i]); }

// Q [0] = Math.pow(slope, 0.5)/n[0]*Math.pow(R [2], 2/3)*A [2];

// System.out.println(Q[0]);

//

// System.out.println(V[0]);

//

// System.out.println(R.length);

// System.out.println(S.length);

// System.out.println(n.length);

// System.out.println(Q.length);

System.out.println("n, S, R, A, Q, V"); System.out.println();

for(int i = 0; i < n.length; i++){

for(int j = 0; j < S.length; j++){ Q [j] = Math.pow(S[j], 0.5)/n[i]*Math.pow(R [j], 0.66666)*A
[j]; V[j] = (1/n[i])*Math.pow(R[j], 0.66666)*Math.sqrt(S[j]); //Q [j]/A [j]; System.out.print(n[i]
```

+", " +S[j] +", " +R[j] +", " +A[j] +", " +Q[j] +", " +V[j]); System.out.println(); } System.out.println(); }

// double $d_50 = 2$;

// double s = 2.7;

// double v = 1.14E-6;

// double $D_s tar = Math.pow((9.81*(s-1)/(v*v)), 1/3)*d_5 0$;

// double w = $v/d_5 0*1000*(Math.pow((10.36*10.36+1.049*D_s tar*D_s tar*D_s tar), 0.5)-10.36)$;

//

// System.out.println(D_star);

// System.out.println(D_star);

// System.out.println(w); }

}